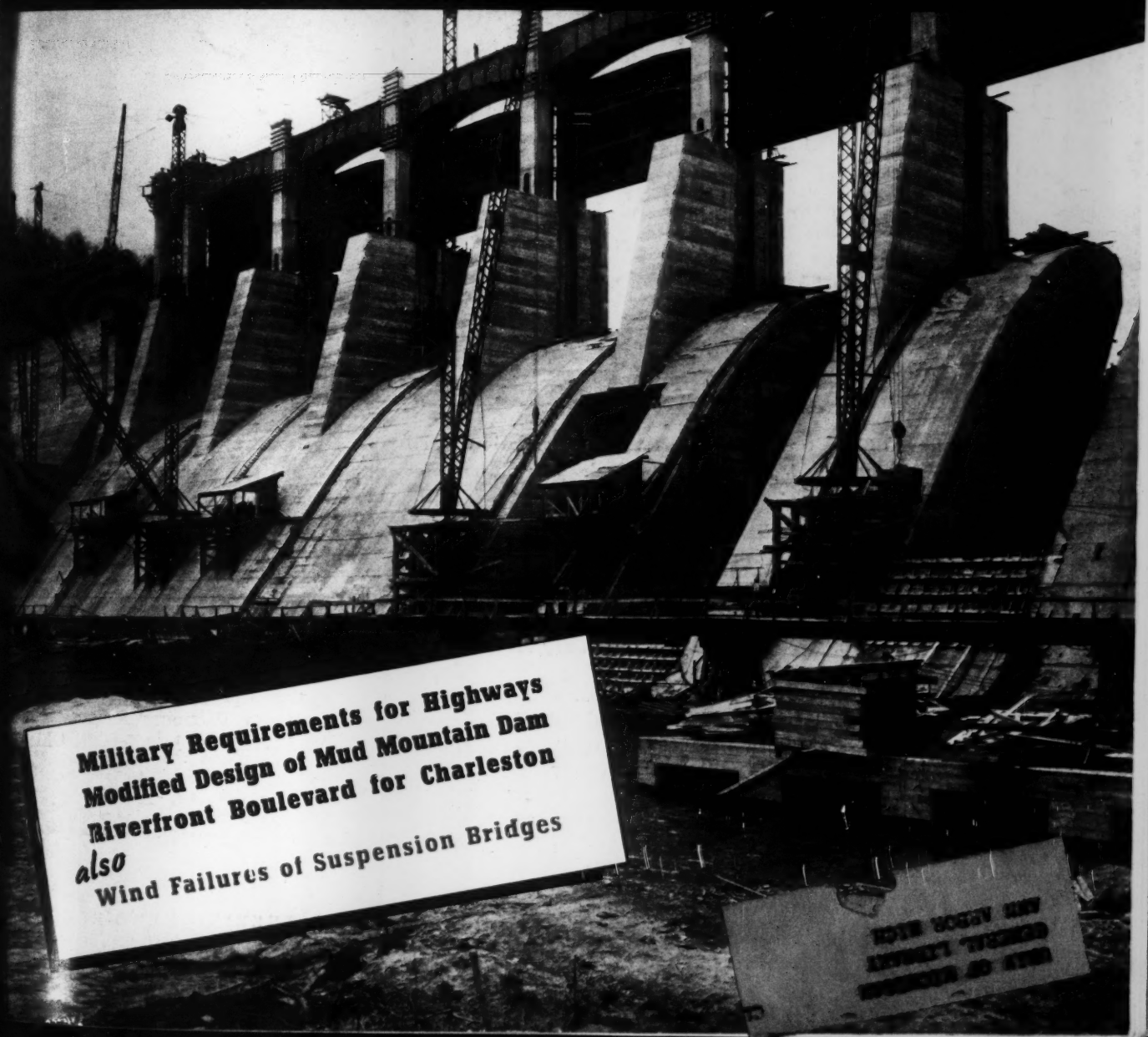
ENGINEERING NEWS-RECORD

MARCH 13, 1941

Difficult foundation conditions met successfully on the Claytor hydro project



Military Requirements for Highways
Modified Design of Mud Mountain Dam
Riverfront Boulevard for Charleston

also

Wind Failures of Suspension Bridges

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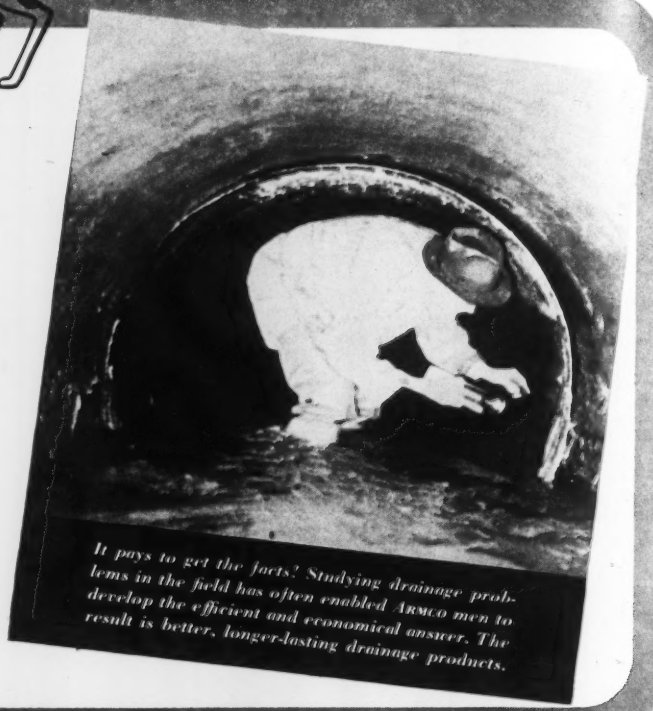

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ENGINEERING NEWS-RECORD

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THIS WEEK

• Both because of the unusual foundation problems which were solved, and because it has the distinction of being one of the few private utility hydro developments to be undertaken during the last few years, the description of the Claytor project in Virginia occupies the lead position in this issue.

• Defense requirements are having a definite influence on highway design and the new specifications developed by the PRA to meet the problems of moving heavy military equipment are reviewed.

• Going back in bridge history to about 1800, Prof. J. F. Finch of Columbia University reviews wind-failures of suspension bridges, and finds a situation which might be referred to as the "Evolution and Decay of the Stiffening Truss."

THINGS TO COME

• Surface cracks which have developed on the concrete of Parker Dam, after only two years of service, have been attributed to adverse alkali-aggregate reactions. A frank discussion of the situation and the investigations which have been made will be the subject of an article by R. F. Blanks of the Bureau of Reclamation.

• Recommendations which have been made by consulting sanitary engineers to the U. S. Army for the adaptation of good sewage disposal practice to the special conditions prevailing in Army camps will be outlined.

• A plate girder of 271-ft. record span is the notable feature of the Main Ave. bridge approach in Cleveland. The foundation design and the erection procedure will be described.

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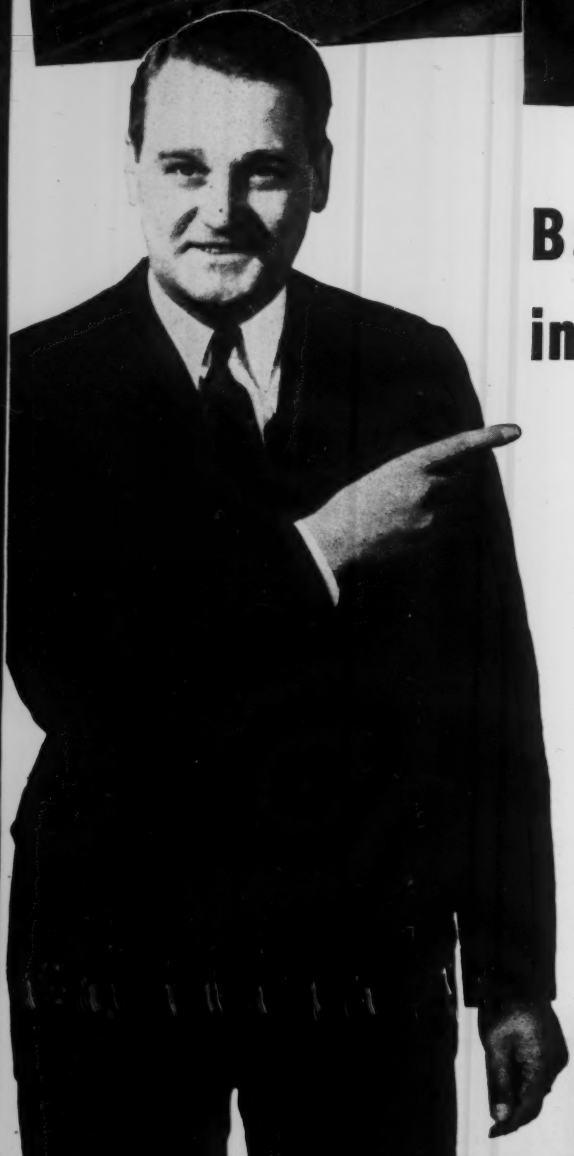

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Tacoma Bridge insurance payments in dispute

Owners ask full value, while insurance companies claim that rebuilding can be done for about one-third of that.

Declaring the collapsed Tacoma Narrows suspension bridge a total loss, the Washington State Toll Bridge Authority March 1, filed a proof of loss with underwriters for $5,200,000, representing 80 percent of its $6,500,000 value, and the full amount of the insurance. Answering the claim, the 23 insurance companies which underwrote the bridge declared in a formal statement that they believe the span can be restored to its original condition at a cost not exceeding $1,800,000.

Ask almost complete dismantling

In making the claim for total loss, assistant state attorney-general Oliver Malm filed under a state law that provides that total loss may be claimed "where the structure insured cease to exist as such by reason of damage to it." It is contended that state statutes would supersede any exemptions or loopholes in the policies themselves by which the insurance companies might conceivably attempt to avoid payment of the full loss. Because the bridge had been in operation only four months, the Authority refused to write off any depreciation.

As bearing on the total loss claim, the Authority's board of consulting engineers reported March 10 that only the piers are definitely salvageable, although there is a possibility that the central section of the towers and the three lower struts may be used. All the remaining superstructure, it is said, must be dismantled before the causes of the collapse can be determined.

The statement by the underwriters was issued by Paul A. Carew, manager of the Marine Office of America in Seattle, and chairman of a committee representing the interests of all companies involved. "Acting upon the advice of eminent engineers of national reputation," the statement said, "it is the definite opinion of the insurance companies that all damages to the bridge can be fully repaired at a cost of not exceeding $1,800,000, and, accordingly, as provided by the insurance policies, a demand for an appraisal to determine the actual amount of damage has been filed with the Washington Toll Bridge Authority."

The "demand for appraisal" means that the insurance companies have invoked the arbitration clause of their contracts with the Bridge Authority. This clause provides that in the event of a dispute over loss, both state and underwriters select an appraiser, and that the two appraisers then select an umpire, to whom they will submit all questions on which they cannot privately agree. The insurance companies named as their appraiser Isaac Farber Stern, consulting engineer, Chicago.

In the event the appraisers fail to agree on an umpire, one would be appointed by a Washington court of record. The purpose of the board of arbitration would be to reconcile the differences existing between the state and the underwriters, and to make a formal finding of loss. It seemed generally agreed last week, however, that the dispute eventually will go to court.

Engineers advising the insurance companies, the statement said, had declared that the towers need not be taken down nor the cables respun. The insurance companies' repair cost of $1,800,000 was based upon use of the towers and cables and the construction of a new deck. Inspecting the bridge last week for the insurance companies were Clifford E. Paine of Chicago, H. D. Robinson and Shortridge Hardesty, New York, Prof. W. M. Wilson, University of Illinois, Clark Clarahan, Chicago, and Prof. Hardy Cross, Yale University.

Cloverleaf aids traffic on strategic highways in Ohio

Completion of this cloverleaf grade separation on U. S. Routes 40 and 42 near Lafayette, Ohio, approximately 22 miles west of Columbus, is another step in the Ohio Department of Highways program to eliminate traffic hazards on all important routes. This improvement, recently completed, is of unusual importance due to the fact that both Routes 40 and 42 are of military significance, the former being a priority military road.

In carrying out the improvement it was necessary to relocate a short section of Route 42 (the diagonal in the illustration) both to the north and south of Route 40 (top to bottom). In addition to constructing the cloverleaf turns and the overhead, it was also necessary to construct 1½ miles of paving to the north of Route 40, and four miles on the south.

L. L. Clymer of Bluffton, Ohio, was general contractor. Including engineering, paving and bridge construction, the project was built at a cost of approximately $300,000.

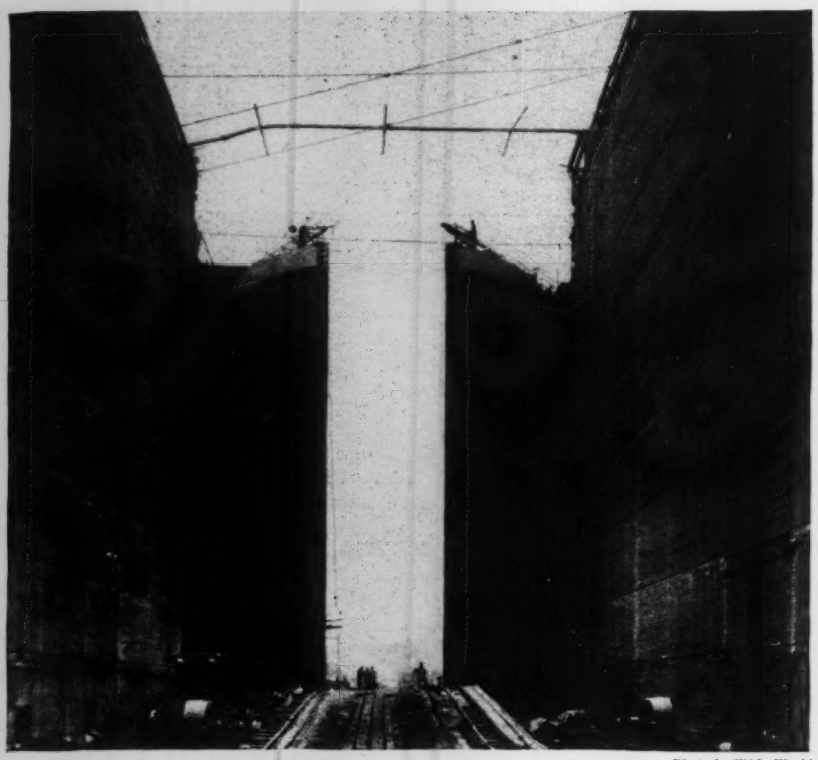


Photo by Wide World

TVA's Watts Bar lock placed in operation

The lock at Watts Bar Dam, on the Tennessee River midway between Knoxville and Chattanooga, was placed in temporary operation March 6, just 53 weeks after the first concrete was placed for the structure.

The lock chamber is 60 x 360 ft., designed for a normal lift of 57 ft. and a maximum lift of 70 ft. The downstream gates are complete and in use but the permanent upstream gates cannot be used until the dam is completed and the reservoir nearly filled. Temporary upstream gates have been installed below the permanent gates to withstand up to a 10-ft. head and permit navigation during completion of the dam.

Watts Bar Dam is scheduled to be finished in Dec. and will provide a 9-ft. channel to Fort Loudoun Dam, 72 mi. upstream. The dam will be 97 ft. high and 2,965 ft. long, with a reservoir area of 41,600 acres.

F. C. Schlemmer is project manager, Oren Reed, construction engineer, and J. S. Lewis, construction superintendent for the Tennessee Valley Authority.

Army Engineers branch notes 100th anniversary

The U. S. Lake Survey on Mar. 3 celebrated "demonstration day" to commemorate the one hundredth anniversary of the founding of that body by an act of Congress in 1841. The Lake Survey is a branch of the Corps of Engineers, U. S. War Department. The first office was opened in Buffalo, N. Y., and was moved to Detroit, Mich., in 1845, where it has since remained.

The primary purpose of the Lake Survey is the publication of reliable navigation charts and related data for all the Great Lakes. Col. Paul S. Reinecke is the present district engineer. He has a staff of 70, of which 30 are in the engineering section, 31 in the drafting and reproduction, and 9 in the administrative section.

WPA gets $375,000,000 for operations to July

The deficiency bill carrying $375,000,000 to continue operations of the WPA to July has been signed by the President. The bill also carries $5,000,000 to provide mobile emergency housing—trailers, converted ships, and the like—in defense areas until regular housing can be built; this is a reduction from the $6,750,000 originally asked for the purpose. It is generally expected, however, that more money will be provided later for this purpose.

About 25 percent of the appropriation, it is expected, will be used for defense projects. Employment is estimated as follows: March, 1,750,000; April, 1,550,000; May, 1,400,000; June, 1,300,000. These figures represent about 22 percent of estimated unemployment.

More Niagara water may be diverted

Diversion of additional water for power generation on the American side at Niagara has been recommended by the Office of Production Management to the State Department. By treaty, American diversion is limited to 20,000 sec.ft. and Canadian to 36,000; Canada recently drew an additional 5,000 ft. with U. S. concurrence. Similar winking at the treaty cannot be used by this country, because the U. S. Constitution makes a treaty supreme law; any taxpayer could obtain an injunction against violation of the treaty. However, machinery is set up in the treaty by which its terms can be modified by resolution of the two legislatures. It is probable that a bill permitting diversion of an additional 5,000-13,000 sec.ft. will be submitted to Congress within a few weeks. Whether this move will be tied to legislation on the St. Lawrence power project has not yet been decided.

Whatever water is diverted can be set immediately to generating power. Niagara-Hudson has a considerable amount of standby capacity resulting from the installation some years ago of new generators to get the maximum power from the limited water available. The less efficient old generators are still usable.

• • •

Load limits reduced on Ohio highways

To aid in safeguarding Ohio highways against crackups due to thaws and heavy traffic, the Ohio Department of Highways, Mar. 2, made mandatory a 25 percent reduction in load limits on about half of the State's road system. The decision was made at a conference of engineers from all sections of the state with Hal G. Sours, highway director, and Harry D. Metcalf, chief engineer, division of maintenance.

• • •

Michigan plans roads by WPA-contract system

Grading and drainage work on unpaved state highways designated as part of the strategic military road system in Michigan will be done by the WPA-Contract system, according to G. Donald Kennedy, state highway commissioner. According to the plan, WPA will contract for the work with private contractors with the understanding labor would be furnished by WPA. This is the first time the state highway department has sponsored a WPA project, and the work will be in addition to the regular highway construction program. According to Mr. Kennedy, the contractors of Michigan have approved the plan.

USHA reverts to advisory agency

Local authorities under U. S. H. A. will now handle design, construction, etc. on defense housing

A change in policy has been instituted as to USHA's share of the defense housing program which has the effect of giving local authorities the same degree of responsibility on defense projects as they have had on the slum-clearance work of past years. Thus the U. S. Housing Authority itself drops its briefly assumed role as a construction agency and reverts to the position of an advisory and financial body.

USHA has been building defense housing under two different laws by two different procedures. Under a bill passed last summer it has been building projects through its local authorities with its own funds. Under the Lanham Act it is building projects itself, with the local authorities acting as assistants and advisors. The former projects have been going forward faster and their method of operation has fitted into USHA's accustomed procedure.

It has been believed, according to FWA spokesmen, that the Lanham Act made it impossible to use local authorities as actual construction agencies, but the agency's legal advisors have now changed their minds.

Effect now is that on all USHA defense housing, the local housing authorities design projects, negotiate and sign contracts, and supervise construction—all under USHA supervision and advice.

• • •

Gate Bridge engineer ousted for testimony

On March 5 Russell G. Cone, engineer Golden Gate Bridge and Highway District, was dismissed by a 9 to 4 vote of the directors of the district, effective March 15, after expressions of resentment by directors that a "subordinate" should have given out information on bridge insurance, on reserve fund for permanent betterments and on possible toll reductions.

The occasion to which the board took exception was a session of the San Francisco grand jury devoted to an investigation of affairs of the bridge. The jury had asked Cone to testify and in complying, under oath, he answered questions and gave opinions concerning these several subjects (mentioned in a news note in *Engineering News-Record* Mar. 6, p. 2).

On announcement of the board's action, the grand jury's foreman released a statement to the effect that such a dismissal is "reprehensible" and liable to intimidate others who may be called to testify before the jury.

Cone's estimate of traffic revenue for the coming year, which constituted a part of his basis for favoring a toll reduction, included expectation of a 4.3 percent traffic revenue increase during 1941. Figures for February, compiled since his testimony, show an increase of 8.7 percent over a year ago.

Headliners

Brig. Gen. Brehon B. Somervell

Comdr. L. N. Moeller

When the Associated General Contractors of America met in Houston, Texas, Feb. 16-17 for their annual convention, headliners were the two engineers shown above. Brig. Gen. Brehon B. Somervell, new chief of the construction division of the Quartermaster General's office, U. S. Army, told of the defense construction program.

U. S. Navy's shore establishment construction for national defense was discussed by Comdr. L. N. Moeller, who has the big job of building the Corpus Christi naval airbase.

Army construction now "on or ahead of schedule"

Under the direction of Brig. Gen. Brehon B. Somervell, 64 percent of the 239 camps, cantonments and ordnance plants being built and proposed for the expanding army were reported Mar. 6 by the War Department as "on or ahead of schedule and 8 percent completed."

Rapid improvement in the status of the projects has been made within the last two months despite the generally unfavorable conditions encountered during the winter season. Exceptional progress between the weeks of Jan. 3 and Feb. 21, materially advanced the status of the entire construction program.

Statistics show that on Jan. 3, 1941, 236 projects were on the program. Of this total, 44 had not been started, 104 were behind schedule, 60 were on schedule, 20 were ahead of schedule and 8 had been completed. On Feb. 21, 239 projects were on the program, of which 13 had not been started, 55 were behind schedule, 129 were on schedule, 24 were ahead of schedule and 18 completed.

• • •

U. S. Engineers ordered from Red River Dam

Oklahoma's red-haired governor Leon C. Phillips, having just settled his long-standing dispute with federal authorities on the Grand River Dam project, has reopened his old state's rights feud with the government, this time over the $56,000,000 Red River Dam on the Texas-Oklahoma border.

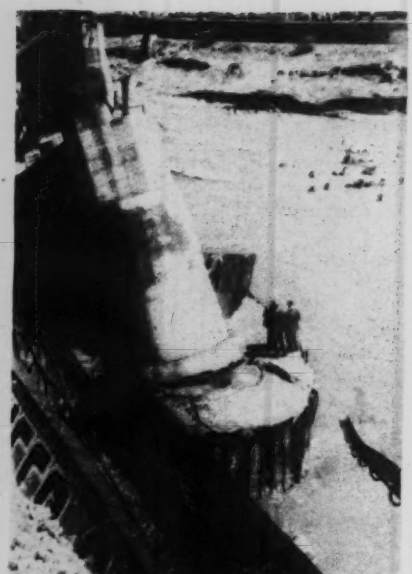
The governor met Maj. R. C. Brown, in charge of land acquisition for the project, and Capt. J. H. Anderson, in charge of relocating highways and railroads, and advised them that under the law state land may not be sold without his consent and, added the governor, "I notify you now as governor of Oklahoma to stay off it until a program of actually repaying us for the losses we will suffer has been worked out."

Governor Phillips had opposed the Red River project since its inception, contending that Oklahoma will suffer most of the damage while Texas will enjoy most of its benefits. Outlet work on the dam is now 85 percent complete.

• • •

Maryland starts defense road program

The initial unit in a $4,285,000 road construction program for the state of Maryland has been started. First work on the program calls for the building of a dual-lane highway to serve the Glenn L. Martin airplane factory at Middle River just out of Baltimore, and will connect with Route 40.



High water wrecks Santa Fe Ry. bridge at Los Angeles

Recent high water in the Los Angeles River, though never of severe flood stage, on Mar. 4 caused the overturning or settlement of one pier of the Santa Fe Railway's main line skew bridge at Los Angeles, with resultant collapse and destruction of a 200-ft. through truss and a 90 ft. through girder span.

Exact reason for collapse of the bridge is being investigated, but possible causes are the scouring of the river bed as a result of channel changes during recent years, the building of a sheet pile bulkhead across the river some distance upstream, which caused the current to swing across the channel toward the side of the bridge pier, or the failure, due to age, of sheet piling protection around the pier. The bridge was originally built in 1901, with the substructure reinforced by sheet piling in 1919.

Plans have been drawn by army engineers for improvement of the channel through this section of the river and were to have been submitted for bid on March 8; the plans had included relocation or reconstruction of this pier. Pending final channel construction, the Santa Fe Railway plans to build a temporary pile trestle at the site.

Detroit's defense plants require new water mains

The building of three large defense plants in Detroit—Ford and Packard aircraft engine factories and the Chrysler tank arsenal—have made it necessary for the Detroit Water Board to add more than 7 miles of new supply lines. According to Laurence Lenhardt, general manager, the installations aggregate: 12,000 ft. of 42 in.; 20,000 ft. of 24 in.; 5,000 ft. of 16 in. and 2,000 ft. of 12 in. All pipe is cast iron. The 42-in. line provides service to both Ford and Packard, while the 24 and 16 in. main connects the Chrysler tank plant; the federal government is participating in the cost of the 3 miles of this line beyond the city limits. The new Hudson plant for naval gun mounts which will be just outside the city limits will also be served from this line to the Chrysler plant. It is estimated that Packard will need 6 to 8 mgd for its Rolls Royce engine plant; that the Chrysler plant will require 2.5 mgd.

In addition to these three extensions several major improvements to the system are being made. Among such jobs is 1600 ft. of 16 in. line for the Chrysler Jefferson Ave. auto plant, and some 6,000 ft. of 8, 10 and 12 in. line for improved fire protection at the Packard auto plant. The latter work is being done by WPA, the remainder by contract.

• • •

Extra lanes for Delaware River Bridge proposed

Opening of two additional lanes on the Delaware River Bridge, between Camden, N. J. and Philadelphia, Pa., has been proposed to relieve traffic congestion due to increasing use of the span, which indicated by a record total of 14,185,835 vehicles having crossed the structure during 1940. This figure represents a gain of 807,600 over 1939.

Construction of two 14-ft. lanes in spaces originally provided for trolley tracks would cost approximately $1,000,000. According to Joseph K. Costello, manager of the bridge, the two additional lanes would be of considerable value in relieving the traffic congestion although the undertaking would not solve the problem until wider approaches on both the New Jersey and Pennsylvania sides are made. Inadequacy of the approaches was emphasized in a recent report by the Delaware River Joint Commission.

• • •

Iowa University starts airport design course

Airport engineering is a special defense course being given this semester by the college of engineering of the University of Iowa. Prof. B. J. Lambert, head of the department of civil engineering, is in charge, and classes are being held at the Rock Island, Ill., district office of the U. S. Engineers.

About 75 engineers are enrolled for the bi-weekly meetings, which began Jan. 6 and will continue until June. Laid out with the aid of H. A. Cook, chief of projects and surveys, C.A.A., the course covers: airports and airways, airport site selection, design of airports, airport construction, operation and maintenance.

• • •

More civilian experts in army construction set up

Gearing itself for highest efficiency in the expectation of a 1941 program even larger than the billion and a quarter dollar job handled in 1940, the Army's Quartermaster Corps has added a design section in which civilian experts play an important role. Under Maj. Hugh T. Casey, chief of design, Frederick H. Fowler, president of the Am. Soc. of C. E., has been appointed chief of the civil engineering section. Assisting him are: E. B. Black on water supply; Samuel Greeley on sewage and refuse disposal matters; A. D. Taylor on site planning; F. S. Poorman on highways and drainage; and C. E. Goldsmith on fire protection.

The design section is part of the engineering branch of the construction division of the Quartermaster Corps. The engineering branch, headed by Col. E. H. Leavey, deals with the general planning and preparation of contracts for projects. It also includes two other sections—architectural under George E. Bergstrom and mechanical under Warren H. McBride. R. H. Shreve is architectural consultant.

Announcement has also been made of the addition of two members to the advisory committee on contracts, formerly composed of Francis Blossom, Forrest Harvey and F. J. C. Dresser. The new members are Brig. Gen. George R. Spaulding and Alonzo J. Hammond. Brig. Gen. Brehon B. Somervell is chief of the construction division.

WASHINGTON HIGHLIGHTS

HE BUREAU OF RECLAMATION is moving pidly ahead on a program of moisture nservation studies—partly doubtless in der to have the work well established the time Congress passes on a per- anent allocation of the work to the griculture or Interior department. nder authority of the reorganization act last summer, Reclamation is organiz- studies of erosion control methods on rigated land, leakage of canal linings, d prevention of reservoir silting. Three gional supervisors to handle the work re appointed last week and a fourth ll be named soon. The new appointees e:

Charles L. Bailey, formerly Secretary d Manager of the Fort Shaw Irriga- n District of the Sun River Reclama- n project, Montana; Thomas W. Parry, rmerly Manager of the Pathfinder Irri- tion District of the North Platte clamation project, Wyoming-Nebras- ; and Hollis Sanford, formerly Asso- te Engineer, Boise Reclamation proj- t, Idaho.

Two bills are now before Congress thorizing moisture conservation work. ey are identical except that one would sign the job to Agriculture, the other to terior.

DRASTIC STREAM POLLUTION control bill th mandatory features has been intro- ced into Congress by Representative rl Mundt. The bill would declare it a blic nuisance to discharge into naviga- waters and tributary streams any ste "injurious to human health or to y form of life, including aquatic life, migratory birds." Injunction suits abatement of the nuisance could be ought in federal court. A new division pollution control in the U. S. Public alth Service would be empowered to standards of purity on each stream. esent polluters would be required to et this standard within two years, ich could be extended five years by division and five years more if the luter has made a sincere effort to elimi- te the pollution.

A much milder bill, without the man- tory features, was put before Congress th administration support in January. st session the same issue arose. Rep. ndt was successful then in having the use adopt his drastic version, while Senate passed the milder measure. e bill died in conference.

E WAR AND NAVY DEPARTMENTS in operation with the Office of Education ve asked Congress to provide $115,- 0,000 for school facilities for more than 200,000 children expected to be located in or near Army and Navy reservations. If this request goes through it will clarify considerably the situation as to the pending $150,000,000 Lanham community facilities bill; Security Administrator McNutt has been making a strong effort to have the major part of the Lanham bill earmarked for construction and operation of schools.

ONE FACTOR behind the recent increase in jurisdictional strikes on construction work is the illness of president John Coyne of the Building Trades Department of the AFL. The machinery for settling jurisdictional disputes which worked so successfully last year depended very much for its effectiveness on Coyne's personality. Crux of the system was immediate temporary decision on each issue as it arose rendered by Coyne himself.

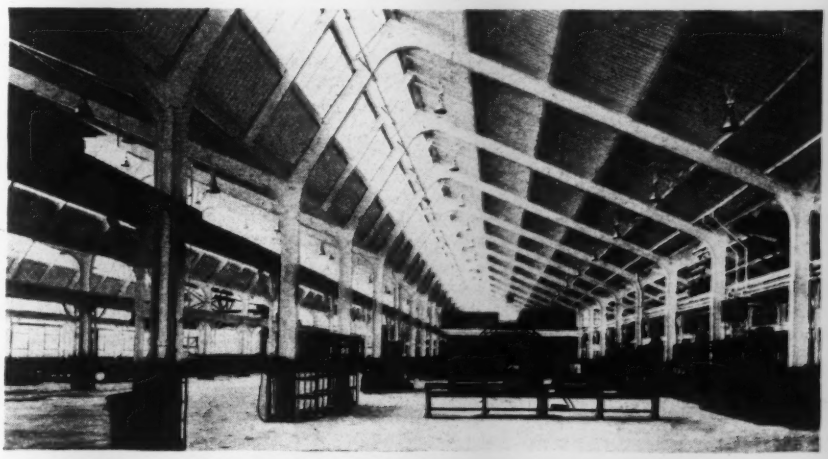
THE ALUMINUM COMPANY OF AMERICA has decided not to build its proposed $37,000,000 Fontana Dam hydroelectric project on the Little Tennessee River. Alcoa reached this decision as a result of the Federal Power Commission's insistence that a license be obtained for the dam; the company is unwilling to subject its project to the 50-yr. recapture clause imposed by the Federal Power Act on licensed projects. Alcoa states that it will cooperate with the TVA or any other government agency that wishes to carry out the project. It is thought likely that TVA may do so.

Last month Alcoa asked permission to withdraw its declaration of intention on the dam—which would have had the effect of wiping out the FPC ruling that a license is required in case the project should be brought up again. FPC refused this permission, commenting that "not even the urgent demands of national defense can alter (Alcoa's) apparent determination never willingly to submit any of its hydro projects to the duly enacted requirements of federal law."

• • •

Consultants named for Chicago water plant

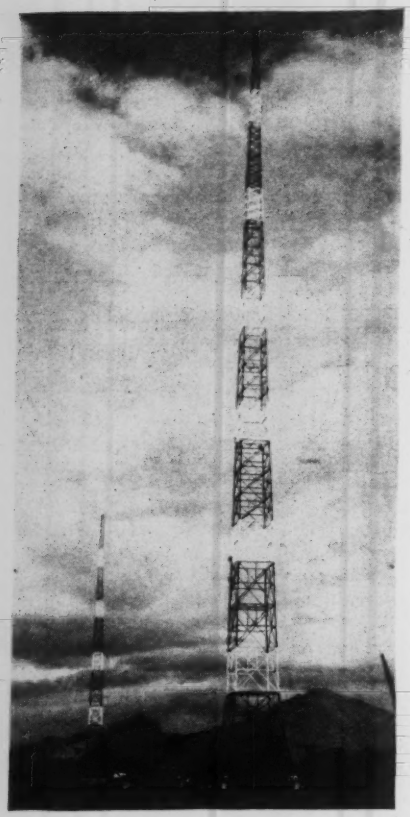
City Engineer W. W. DeBerard of Chicago obtained authority from the city council March 5 to engage Louis R. Howson and Paul Hansen, of Chicago, to advise on plans for completion of the South Side water filtration plant now being built at 79th St. and Lake Michigan. The substructure of the plant is now nearing completion under two contracts held by M. Pontarelli & Son, Inc. General plans for the superstructure and operating equipment have been made by the city engineering department, but many details are yet to be worked out.



Shadowless sawtooth construction for new Vickers plant

Sawtooth roof construction of welded rigid frame design, which eliminates the shadow-producing truss members of more standard construction, provides five 40-ft. aisles in the well lighted area of Vickers, Inc.'s new plant recently completed in Detroit, Mich. The area of the new plant totals 93,000 sq. ft.

The new plant will be devoted to the assembly of hydraulic pumps, valves, controls and transmissions. Lockers, wash-up and toilet facilities have been provided on a mezzanine at a central location in the middle aisle, leaving the other aisles free for operation of 5-ton electric traveling cranes. All service piping and busways have been nested between the columns and crane rails from which unit heaters have been suspended. This arrangement leaves the aisles entirely without obstruction from floor to roof. High intensity mercury vapor lamps installed throughout the plant are mounted with chains which permit lowering of the individual units for cleaning. The lighting never falls below 20-ft. candles intensity. The Austin Co. are designers and builders.



Radio tower designed for 100 mile wind

Designed to resist a 100 mile wind, an unguyed steel tower, 400 ft. high, was recently completed at Phoenix, Ariz. for radio broadcasting station KTAR.

Electrical requirements called for a design with a minimum taper of 5 percent. By spreading the two lower panels, stresses are reduced to safe limits for the insulator bases, which are of low crushing strength. The top dimension is 18 in. and the spacing between insulators at the bottom is 32 ft.

The tower was erected in 18 days at a cost for fabrication and erection of $14,000. The Allison Steel Co. of Phoenix, Ariz. was the contractor.

Sanitary engineers to hear of defense role

Sanitary engineers in the New York metropolitan area will have an opportunity to hear of federal, state and local plans concerning sanitation aspects of national defense at a symposium to be held at Manhattan College, Friday evening, March 21. Participating in the program are Arthur P. Miller, district engineer, U. S. Public Health Service, A. F. Dappert, principal sanitary engineer, New York State Department of Health, and Seth Hess, chief engineer, Interstate Sanitation Commission.

JOBS OF THE WEEK

BOMBER ASSEMBLY PLANT, Tulsa, Okla.

Manhattan Construction Co., Muskogee, Okla., and Long Construction Co., Kansas City, Mo., have received a joint contract for construction of an aircraft plant for the U. S. Engineers, Washington, D. C. The work, estimated cost of $10,476,400, is being done on a cost-plus-fixed-fee basis. Consolidated Aircraft Corp., San Diego, Calif., and Douglas Aircraft Corp., Santa Monica, Calif., will operate the plant.

BOMBER ASSEMBLY PLANT, Fort Worth, Tex.

Austin Company, Cleveland, O., will build a bomber-assembly plant for U.S. Engineers, Washington, D. C. Total estimated cost of the work is $10,000,000. Consolidated Aircraft Corp., San Diego, Calif., will operate the new plant.

BOMBER ASSEMBLY PLANT, Kansas City, Kan.

G. L. Tarlton Contractor, Inc., MacDonald Construction Co. of St. Louis and S. Patti Construction Co., Kansas City, Mo., were awarded a joint contract for the construction of a bomber-assembly plant. Construction alone will cost $3,706,284 and the total cost will be $10,000,000. U. S. Engineers, Kansas City, Mo., are in charge of construction. North American Aviation, Inc., Inglewood, Calif., will operate the plant.

AIRPORT, Fort Wayne, Ind.

Cooke Contracting Co., Detroit, Mich., has received the contract for clearing and grading and construction of the runways, taxi strips, and aprons at the Fort Wayne airport. The $1,393,492 cost will be shared by the War Department and the Civil Aeronautics Authority.

DREDGING and PIER CONSTRUCTION, Jamaica, B. W. I.

Frederick Snare Corp., New York, will construct the temporary shore facilities, do the dredging work, and build the pier at Naval Air Station. Bureau of Yards and Docks awarded contract for the work, which is expected to cost $2,300,000.

AIR BASES On British Possessions

Arundel Corp. and Consolidated Engineering Co., Inc., both of Baltimore, will do the dredging work and construct the pier and shore facilities at St. Lucia and Antigua, B.W.I. air bases. This work is estimated to cost $3,810,000. The contractors will also purchase, assemble and fabricate materials for the Naval air stations at Newfoundland, Bermuda, Bahamas, Jamaica, Antigua, St. Lucia, Trinidad and British Guiana under terms of a $1,004,000 contract. The Bureau of Yards & Docks, Navy Department, awarded the contracts.

AIRBASE, Macon, Ga.

MacDougald Construction Co., Griffin Construction Co. and Nonnemaker-Clayton Co., all of Atlanta, Ga., will build flying school No. 1, near Macon at a cost of $2,094,735. The contract is on cost-plus-fixed-fee basis. The War Department awarded a contract and also a collateral contract to J. B. McCrary Engineering Corp., Atlanta, for the architectural and engineering services.

PIPELINE, Louisiana

B & M Construction Corporation, Oklahoma City, will build a 135-mi. pipeline from Monroe, La., to Logansport. Estimated cost of the work is $2,200,000. The Southern Natural Gas Co., Birmingham, Ala., is owner of the line.

DIVERSION TUNNEL and DAM, New York, N. Y.

George M. Brewster & Son, Bogota, N. J., has been awarded the contract for constructing the diversion tunnel and cofferdam at Neversink Dam in Sullivan County, N. Y., for the New York City Board of Water Supply in accordance with a bid of $3,406,895 opened Feb. 18.

SHIPYARD, Baltimore, Md.

Booth & Flinn Co., Pittsburgh, Pa., has been awarded a contract by Bethlehem-Fairfield Shipyard, Inc., Baltimore, for constructing the shipyard work estimated to cost $3,000,000. Total estimated cost of the project is $7,836,000.

NOTE—Additional bidding and contract news on over 800 projects large and small appear in the Construction News section beginning on page 163.

OBITUARY

NRY CLEVELAND LAMB, partner in rm of Elwyn E. Seelye & Co., con- engineers, New York City, died ite Plains, N. Y. Mar. 6. He was e was a graduate of Cornell Uni- y.

ES McFARLAND YOWELL, engineer e Mississippi State Highway De- ent, died Feb. 25 at McComb, Miss. owell was an engineering graduate University of Mississippi.

G. GEN. W. A. MITCHELL, U. S. (retired), of Arlington, Va., died 6, at Tuscambia, Ala. Gen. Mitchell rofessor of civil and military en- ring at the U. S. Military Academy st Point from 1922 to 1932.

T. HAMBLEN, civil engineer of Fort . Tex., died Feb. 28. He was 41 old and a graduate of Texas College.

ARLES FRANKLIN GREGORY, 56, of Liberty, W. Va., assistant superin- nt of the West Virginia Department ighways, died at Youngstown, Ohio, 28.

ARENCE H. BILLITER, disposal plant eer, 69, died at Hudson, Ohio, 2.

LLIAM BEVINS, of Hartford, Conn., ormerly superintendent of streets, Mar. 2. Mr. Bevins joined the Hart- street department in 1898, and re- in 1938.

G. GLADNEY, engineer with the ssippi State Highway Department, 50, died Mar. 1 at Jackson, Miss. Mr. Gladney was formerly dean of the school of engineering at the University of Mississippi.

WILLIAM H. GARDINER, of Seattle, Wash., 57, vice president of Winston Bros. Construction Co., of Minneapolis, Minn., died Mar. 6 at Louisville, Ky.

CECIL C. WILLIAMS, engineer-manager for the Wenatchee reclamation district since 1910, died Feb. 24 at Wenatchee, Wash.

CONTRACTS AND CAPITAL

ENGINEERING CONSTRUCTION awards for the week, $132,626,000, more than tripled the volume for the corresponding week last year, and topped last week's award figure by 28 percent. This is the 28th consecutive week in which current awards have exceeded their respective 1940 values.

Public construction is at the third highest peak of the year, 292 percent over last year, and 36 percent higher than last week. Private awards are 40 percent above a year ago, but are 3 percent under a week ago.

The current week's total brings 1941 construction to $1,256,538,000, a 130 percent gain over the 11-week 1940 period. Private and public construction are 91 and 151 percent higher, respectively, than in the period last year.

In the classified construction groups, gains over last week are reported in waterworks, sewerage, commercial building and large-scale private housing, public buildings, streets and roads, and unclassified construction. Increases over the 1940 week are in waterworks, sewerage, industrial and public buildings, and unclassified construction.

New capital for construction purposes for the week totals $5,282,000, a drop of 83 percent from the volume for the 1940 week. The current week's new financing is made up of $4,117,000 in state and municipal bond sales, $1,135,000 in corporate security issues, and $30,000 in RFC loans for public improvements.

New construction financing for the year to date, $693,264,000, tops last year's 11-week volume by 220 percent.

CONTRACTS

(Thousands of dollars)

	Week Ending Mar. 14 1940	Mar. 6 1941	Mar. 13 1941
Federal	$2,537	$59,994	$84,323
State & Municipal	26,023	22,545	27,620
Total public	$28,560	$82,539	$111,943
Total private	14,748	21,423	20,683
TOTALS	$43,308	$103,962	$132,626
Cumulative 1941 (11 weeks)			$1,256,538
1940 (11 weeks)			$545,521

Note: Minimum size projects included are: Waterworks and waterways projects, $15,000; other public works, $25,000; industrial buildings, $40,000; other buildings, $150,000.

NEW PRODUCTIVE CAPITAL

	Cumulative 1940 11 weeks	1941 11 weeks
NON-FEDERAL	$216,747	$184,264
Corp. Securities	34,186	57,304
State & Mun.	93,894	87,539
U.S.H.A. loans	88,022	13,521
R.F.C. loans	645	25,900
FEDERAL		$509,000
TOTAL CAPITAL	$216,747	$693,264

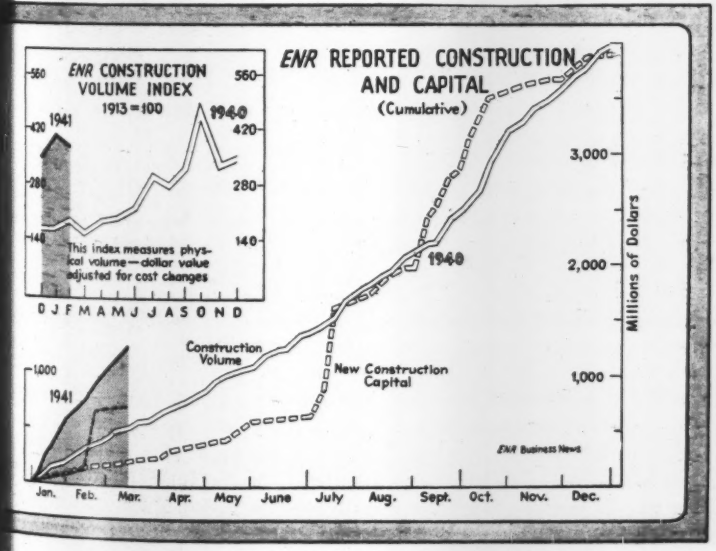
FHA MORTGAGES

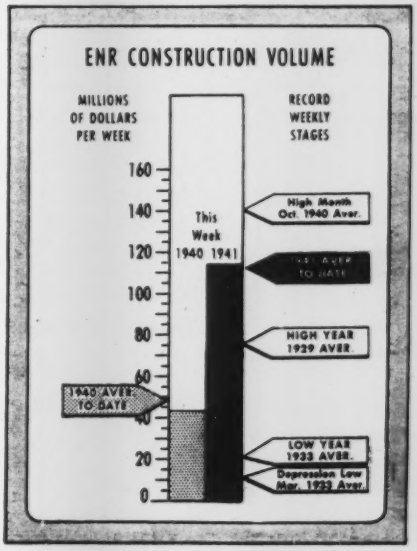
	Week Ending Mar. 9 1940	Mar. 1 1941	Mar. 8 1941
Selected for appraisal	$26,556	$26,182	$30,561*
Cumulative 1941 (10 weeks)			$225,280*
1940 (10 weeks)			$186,425

* Subject to revision.

ENR INDEX NUMBERS

Index Base=100		1913	1926
Construction Cost	Mar. '41	250.71	120.52
Building Cost	Mar. '41	207.88	112.37
Volume	Feb. '41	374	164





Comment and Discussion

Readers' opinions on matters that concern engineers and contractors

Arch Roof Hangars

Sir: In your issue dated Oct. 24, 1940, p. 551, you mention arches being built under license from the "Steel Arch Roof Construction Co." This is incorrect. The correct name of our firm is Arch Roof Construction Co., Inc.

We have designed all of the hangars for Westover Field at Chicopee Falls, Mass.; all the hangars at MacDill Field, Tampa, Fla.; and all the hangars at Elmendorff Field, Anchorage, Alaska. These are for the Constructing Quartermaster of the War Department.

L. DAVIDSON
President, Arch Roof Const. Co., Inc.
New York, N. Y.

Future of Work Relief

Sir: Since the inception of work relief early in the Roosevelt administration the writer and his partners' clients have made extensive use of this type of labor for practically all types of municipal construction with results in general satisfactory from the standpoint of the municipalities and the federal government.

The question arises continually in my own mind, "Will WPA or some other form of work relief be a permanent necessity for our civilization?" In order to make even an attempt to answer this question, we should analyze some of the basic causes for the lack of private employment for this class of people. Among these causes can be listed the following:

1. Industrial mechanization has progressed to a point where the factory demand is preponderantly for skilled or at least semi-skilled labor. Where unskilled labor is required employers seek to employ only the most alert and physically strong.

2. Conduct of public works construction by private operation has closely followed the industrial mechanization and there also only the strong and skilled are favored for employment.

3. Freezing of wage scales at a single and high level, especially on unionized operations, eliminates demand for men of medium skill and strength.

4. A pronounced slackening is noted in private heavy construction where a large part of the unskilled labor ordinarily is employed. This tendency is most commonly ascribed to a lack of investment capital formerly furnished by the wealthy element of the population. High income taxes now have tended to dry up this source of investment funds.

5. Accidents of birth, the various weakening infectious diseases, alcoholism and many other causes insure that society will always have a considerable percentage of men and women of only medium alertness and strength. Provided high mechanization and organization of industry and construction work continue to preclude the employment of this class of people, then employment must be solved through WPA or allied lines of directed employment.

To date there have been sufficient worthwhile and serviceable public projects to employ WPA workers near to their places of residence, but the amount of this type of work is rapidly decreasing.

The only answer seems to be the reduction of the WPA load by stimulating reemployment of many of these workers in productive industry, and the extension of WPA activities to new forms of public construction.

How can these workers of medium alertness and strength be returned to and utilized in industry? I would suggest the following:

1. Expansion of industry so that all skilled men would be employed in positions requiring skill, making way for absorption of relief workers in secondary positions now in many cases held by potential skilled workers.

2. To develop capital for industrial expansion, encourage formation of supervised "investment associations" similar to the present "Building and Loan" associations. These will be necessary if present income tax and profit laws are to be retained, otherwise there will be no adequate source of capital for this expansion. Liberalize laws for investments of insurance companies, trust funds, etc., to allow more investment in stocks preferably of one-class stock companies without large bonded debts. Make a small proportion of this type of capital available for investment in new enterprises.

3. Revise tax laws so that industrial enterprise may spread its profits and resulting tax payments over at least a 5-year average.

4. Develop means to educate the workers themselves to invest their savings in industrial enterprise. At present society, rightfully or wrongfully, is in a majority against having a wealthy class furnish this capital.

5. Give governmental financial assistance to manufacturers of stable articles to build up inventories in slack and off-season periods and thus assist in stabilizing industrial employment. This would in turn help stabilize commercial (merchandising and distribution) employment, enable the workers to have higher annual earnings and to furnish more of the new capital constantly required.

The above suggestions are offered not in the sense of a specific cure-all but to help start discussion toward constructive methods of preventing further private enterprise stagnation. The situation will become even more critical from an employment standpoint when the present artificial and in reality unpaid-for rush of defense orders is completed.

GEORGE O. CONSOER
Consoer Townsend & Quinlan
Consulting Engineers, Chicago

First Contractor-WPA Set-up

Sir: In your issue of Dec. 5, 1940, p. 737, there is a communication from D. C. Greer, State Highway Engineer, of Texas, which states in part:

> The Texas State Highway Department was, in so far as we have been able to learn, the first public body in the United States to inaugurate contract projects in which WPA participated. The Texas State Highway Department, late in 1938, commenced a study of contract procedure as it could be applied to projects in the construction of which WPA facilities could be used to advantage.

The purpose of this letter is to correct that statement, as the New Jersey State Highway Department claims to have originated the contract-WPA method of performing work as a sponsor with the WPA in the early part of 1938, and in fact between

April and July of 1938, had under construction eighteen WPA contracts. Since then the New Jersey State Highway Department has pursued this method consistently as part of the means of performing work with WPA labor on the New Jersey highway system as well as in cooperation with the various counties and municipalities wherever the state highway department allotted state funds to those municipalities as aid, which was used as sponsors' funds.

JAMES LOGAN
New Jersey State Highway Engineer
Trenton, N. J.

Military Roads

Sir: That some good can come of war is hard to believe and yet I remember that once years ago, I was traveling through southeastern Missouri near Pilot Knob and ran across a fine road. This being so unusual, I asked a native "how come?" "Oh," he said, "the Union Army built that. They built it to run away from Marmaduke on."

N. F. HOPKINS
Pittsburgh, Pa.

Athens Water Supply

Sir: Recently I have been reading that interesting book, "The Story of Tunnels" by Archibald Black, published by McGraw-Hill.

I understand that this book is written for the lay reader and that it entailed a great deal of work in collecting the necessary data. Inasmuch, however, as some people are liable to take this book too seriously, I venture to correct one inaccuracy that has crept in. I might even add some interesting details.

Mr. Black has a story about the Hadrian Aqueduct on p. 11. Ulen & Co., in 1925, had two contracts with the Greek government, one for a temporary supply which included repairs to the Hadrian Aqueduct, and another for an entirely new supply after the modern manner, the so-called Marathon supply.

Mr. Black says, "In 1925 the American firm of Ulen & Co. was called in to partly reconstruct the old tunnel and to connect it to an entirely new tunnel which was also to be built by the American engineers. So the old tunnel of the Romans is again serving as an essential part of the water system of Athens . . ."

As a matter of fact, the Hadrian Aqueduct is not connected in any way with the new supply except perhaps through some cross-connections in the distribution system. The Hadrian Aqueduct is very small, about a meter high and a meter wide, and the new tunnel has an area something like ten times larger. The new distribution system delivers the water to the city from the new Boyati tunnel at a considerably higher elevation than the Hadrian Aqueduct. The new system includes a complete distribution system, reservoirs and a modern purification plant.

The story about the new Boyati tunnel being connected with the old Hadrian may have originated in a booklet published by the Hercules Powder Co. in 1931. In this booklet the tunnels are shown as connected and it is stated that the information was supplied by R. H. Keays (the writer). I never supplied any information directly to the Hercules Powder Co.

The error probably arose from a misinterpretation of some letters of mine at that time in which there was a story about how the rather large quantities of water that were met during the construction of the Boyati tunnel were being temporarily pumped into the Hadrian Aqueduct to eke out the scanty supply of Athens until the new Marathon supply was ready. The route of the Hadrian Aqueduct crossed a small valley close to the Athens end of the Boyati tunnel and it was quite simple to clarify the tunnel water and pump it into the Hadrian Aqueduct. No one ever drank Hadrian water in recent times, I believe.

So the story grew. Some engineer-reader of Mr. Black's book might think it a rather dumb thing to do what he says was done, so it would be a good thing to correct before it becomes history.

R. H. KEAYS
New York, N. Y.

Vertical Curve by Radius

Sir: In *Engineering News-Record* of Jan. 2, 1941, p. 9, H. Elliott Brown mentions as unique the practice of designating a vertical curve by radius computed from the following approximate formula:

$$\text{Radius} = \frac{\text{Length of V.C. (ft.) x 100}}{\text{Change (percent) of grade}}$$

As Mr. Brown implies, this method is probably not widely used, nevertheless, it is by no means new. On the San Francisco Municipal Railway, we have used this formula since the beginning of the system nearly 30 years ago, and so far as I know, it may have been handed down to us from the older systems of this city.

In our practice we first use the above formula to determine a circular curve that will fit the pavement fairly well, and whose radius is not less than the minimum permitted for car clearance. The curves are generally short, 50 ft. or less in length, and in most cases the girder rails used require shop bending. When this is necessary they are curved to the radius computed from the formula.

In establishing the grade elevations the curve is treated as a parabola instead of a circle but in this class of work the difference between a parabolic arc and a circular one is not great enough to cause any difficulty.

L. V. DEGNAN
Railway Engineer
Public Utilities Commission
City and County of San Francisco

Maas Tunnel Progress

Sir: I am glad to inform you that construction on the Maas Tunnel has not been delayed very much by war circumstances and that the last of nine river caissons was sunk in December, 1940.

J. P. VAN BRUGGEN
Chief Engineer of the Tunnelworks
Rotterdam, Holland
Jan. 3, 1941

The above letter from Mr. Van Bruggen came in response to an inquiry as to progress on the tunnel sent to him in December with copies of the issue of *ENR* containing his article on the project.—EDITORS.

Chain-Store Engineering

A man who states that he is not a graduate civil engineer but has done a lot of engineering work for the county commissioner of his county which has been so successful that he has been called on for more information, writes to the McGraw-Hill Book Co., saying, "I have been wondering if you issue a general catalog on all books published by you. I have been able to get some scientific books from Montgomery Ward and Sears Roebuck & Co., although they do not carry all books that a person might need."

EDITORIALS

VOLUME 126 NUMBER 11

March 13, 1941

Forgotten History

IT IS A VALUABLE ADDITION to suspension bridge literature that Prof. Finch contributes to this issue in his article reviewing the wind failures that have plagued this type of structure during the past 125 years. For, as the article makes plain, the "failure periods" have been cyclical, corresponding closely to reductions in the amount of longitudinal rigidity. When adequate stiffening was used, failures did not occur; when compromise with stiffening was popular, spectacular collapses increased. Furthermore, in nearly every case the failure was accompanied by vertical undulations similar to those preceding the Tacoma Narrows collapse and noted on other modern bridges. Clearly, the past decade has been a period of compromise with stiffening, and the difficulties encountered might have been avoided if history had not been forgotten. On the other hand to have provided conventional stiffening trusses adequate to resist all of the forces now known to be operative would have made some of our modern bridges too expensive to build. Here is where the worth of present interest in aerodynamic research becomes justified. It may well be that such expedients as storm cables, brakes and streamlined cross-sections may permit lighter and therefore less costly stiffening systems. But in one way or another longitudinal stiffness must be provided. History shows that to compromise with stiffness is to compromise with safety.

Under the Big Top

ONE OF THOSE THINGS that make the construction industry such a gold mine for the Sunday supplement writers was the announcement recently that Mud Mountain Dam, near Tacoma, will be built under a tent. Here is quite enough information for a thrilling piece entitled "The Circus Comes to Construction," but engineers and construction men will see in the announcement a much deeper significance. Mud Mountain Dam is to have a large core of rolled earthfill, and a tent is part of the field man's answer to the soil expert's order that an "optimum moisture content" must be maintained in the fill material; the other part of the answer is to dry out the material initially in oil-fired drums. In one respect introduction of weather protection and scientific drying in earth-dam building is comparable to the introduction of scientific water control in concrete mixing. For some time after the water-cement ratio law was established by concrete technicians, field men honored it by putting in "about" the amount of water called for. Some improvement over wet mixes resulted, but it was not until account was taken of the amount of moisture in the aggregate, supplemented by accurate water measuring tanks, that job concrete approached the strength and uniformity of the laboratory product. Earth dam construction, as planned at Mud Mountain, is a first step to get laboratory results on the job in this field. It is an experiment well worth watching.

New Use for Old Products

TWO DEVELOPMENTS during construction of the Claytor hydroelectric project (p. 388) show how old products continually are being put to new uses. The first opportunity to use a familiar material in a new way arose when the clay overburden at the north abutment of Claytor Dam showed excessive permeability. To seal it off, a sheeted trench was carried down to rock and filled with a sand-asphalt mortar up to water level in the reservoir. This material has the tightness of concrete and the flexibility of graded and tamped clay and is said to have cost less than either to place. Later in the construction program extensive grouting of the foundations was required. Here, it was found possible to utilize the fines, wasted in the aggregate processing, to produce a very satisfactory grout, with a considerable saving in cement. Both of these special applications of old materials serve to show that no construction job is routine; each brings its special challenge and each produces some new or improved method that cuts costs or saves time.

Salvage from Sewage

AN EXTRAORDINARY STORY of byproduct recovery from sewage is told in this issue, wherein is revealed for the first time the commercial phases of disposal operations at Bradford, England. This plant now is producing 500 tons weekly of high grade grease—an invaluable contribution to Great Britain's war effort—and 30,000 tons yearly of fertilizer. Bradford's salvage efforts arose from the exceptional nature of its sewage, which carries immense quantities of lanolin fat; the latter derived from washing wool, of which one-fifth of the

world's production is handled at Bradford. Few, if any, cities in this country could reclaim grease from sewage on a scale comparable with Bradford, but there may be opportunities for small-scale emulation. The recovery of other byproducts from sewage treatment operations—principally gas and fertilizer—is well established in American practice. Salvage from sewage may assume greater significance with increased tempo of our defense program.

Roads for the Army

SPECIFICATIONS FOR HIGHWAYS to be used by the Army, as developed by the Public Roads Administration (p. 399), call for no radical departures from standards now widely used by the state highway departments on the federal-aid highway system. Tanks and other heavy military equipment are being built to avoid concentrations of heavy loads; consequently any highway built for H-15 loading can carry such equipment safely. Only at bridges is there any need for special restrictions as to tank movements.

The major change in the specifications is in shoulder widths, which are required to be much wider than is now common practice; or, as an alternative, widened turnouts can be built at frequent intervals to permit military convoys to park clear of the traveled way. Building such shoulders will add materially to construction costs in many parts of the country and may call for some special consideration on the part of Congress, as they can hardly be set down as a normal requirement for non-military traffic.

Indirectly, the new military highway specifications may have the effect of nullifying the limits on truck loadings now enforced in some of the states. Obviously, the federal government is not going to build highways into military camps and to new defense plants that can safely carry 15-ton trucks and then have the states where they are located stop trucks at their borders because of restrictions on truck loads. Heavy trucking at one time did much damage to our highways, but that time is largely past. Most of the restrictions that remain are not on the statute books to save highway pavements but to reduce competition between trucks and railroads. Removal of the restrictions would be a good thing all around.

The new specifications, therefore, are seen to be broadly constructive. Providing the definite guide long needed for building roads to military requirements, they will begin to show their value as soon as Congress appropriates the funds that have been requested.

Shortage of Engineers

THERE IS GREAT NEED for coordination between the U. S. Office of Education and those who are administering the Selective Service Act if the reported shortage of engineers is not to become more serious as the defense effort increases in intensity. Last week a number of engineering educators met in Washington to discuss with executives of the U. S. Office of Education plans for speeding up the training of students now near the end of their engineering courses. At the same time, draft boards were calling young engineers and engineering instructors into service and assigning them to infantry or other units where the value of their engineering training will be wasted.

Responsibility for this anomalous situation does not rest on local draft boards, who must follow the letter of their instructions, but on the policy-forming officers of the War Department in Washington, who could relieve the possibility of a shortage of engineers by a few simple rulings.

First, there should be a decision as to whether engineering instructors and some or all engineers in training should be exempt from the draft. In Canada, young engineering educators are exempt because what they are doing is recognized as an essential war service. Then it should be decided whether students of civil, mining, chemical and electrical engineering should be switched to mechanical engineering wherever possible, as that is the field where the shortage is most acute. Finally, young engineers who are drafted should be assigned to engineer or shop regiments where they will be most useful and from where, in an emergency, they can be assigned to industries badly in need of engineers.

Our whole schedule of engineering training should not be upset until it is known how real the shortage is and how much it can be reduced by changes in administering the Selective Service Act. Agencies in Washington now working in separate channels should get together.

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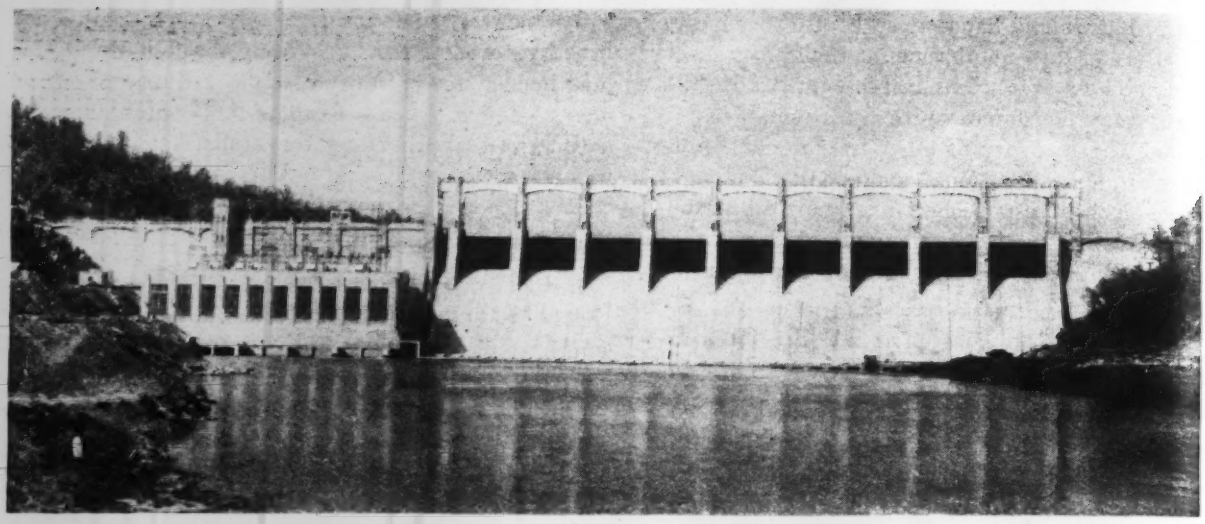


Fig. 1. Claytor hydroelectric development on the New River in Virginia. The discharge passages of the draft tubes under the powerhouse at the left are turned slightly toward midstream to deflect the flow into the main channel of the river.

Design and Construction Features Of the Claytor Hydro Development

Philip Sporn
Vice President and Chief Engineer

H. A. Kammer
Construction Engineer

American Gas and Electric Service Corp., New York

***Contents in Brief**—Construction of a hydroelectric plant to round out the Central System of the American Gas and Electric Co. was begun in 1937 and completed in 1940. Because of the unusual structure of the underlying rock and the abutments an elaborate program of grouting and foundation consolidation was undertaken. In one abutment a sand-asphalt cutoff was employed. When no sand was found near the site, fines for the concrete were successfully processed in a local quarry. In some of the exposed concrete the cement content was varied to study the effect of weathering. Special attention was given to construction joints.*

THE CLAYTOR hydroelectric power development of the Appalachian Electric Power Co., recently completed and placed in operation, is located on the New River in Virginia, about four miles south of the City of Radford. It is the third development of the company on that river: the other two, located further upstream and known as Buck and Byllesby, were completed during the year 1912. A substantial portion of the water rights for the Claytor development were acquired by a predecessor, the Appalachian Power Co., whose properties were merged with others to form the Appalachian Electric Power Co. in 1927.

The welding together at that time of these properties and others north of the Ohio River to form the present central system of the American Gas and Electric Co., together with the building of an extensive 132-kv. transmission system interconnecting a group of low-cost steam generating stations and other energy resources, made impossible immediate economic development of the site under discussion. Besides, it was found necessary to undertake certain hydrologic and geologic investigations in respect to the Claytor development as mentioned later. Before they were completed the 1930 depression with its drastic reduction of power demands had set in.

By the early part of 1937, however, conditions had materially changed: the load of the company had sufficiently increased and was increasing at a rate that made possible attractive utilization of additional hydro-generating capacity in combination with the principal energy sources—the strategically located steam plants. The competitive economics of the Claytor development also was favored by the fact that by that time all of the water rights had been acquired. These facts, together with the threat of inflation or at least of materially higher commodity prices (the latter have, incidentally, since been realized), combined with sharply rising loads confronting the system, resulted in a decision to start immediate construction. Work was consequently embarked upon as soon as practicable thereafter.

The water impounded by and utilized at the Claytor Dam drains from an area of above 2,400 sq. mi. The calculated average flow of the river at the site was about 3,280 cfs. over the 23-yr. period from 1907 to 1930. Investigation had shown, both from the economic standpoint and from the viewpoint of the effect of the reservoir elevation on the Norfolk & Western R. R., whose tracks cross the river at the upper end of the reservoir, that the maximum height of the dam should be such that it would impound the water to about El. 1,846. This is about 115 ft. above the average river elevation existing before construction of the dam. On the basis of available streamflow records, with a pond level normally at El. 1,846 and an installation of 75,000 kw., the annual energy output to be expected is 190,000,000 kwh. for an average year; 400,000,000 kwh. for the best year of record, and about 110,000,000 kwh. for the worst year of record. The storage capacity of the reservoir is about 12,000,000 kwh. between El. 1,846 and that elevation to which the reservoir can be drawn without undue sacrifice of station capacity. Considering the type of load served by, and the capacity of the already existing steam-generating stations of the Appalachian Electric Power Co., it was decided to develop the Claytor project neither strictly for peak nor for relatively firm base operation. Four Francis-type turbines were installed, each having a capacity of 26,000 hp. at 110-ft. head. This makes reliably available 35,000 kw. at 25-percent load factor all of the time, with an additional 20,000 kw. at 25-percent load factor about 90 percent of the time, and the balance available is determined by river flow.

Fig. 3. The normal method of stage construction was employed in the river, cofferdams being built out from the two banks for the first stage.

During the period from 1927 to 1935 an extensive exploration program had been carried out to determine the tightness of the reservoir rim and to decide upon the best location for the dam, so that when it was decided to proceed with the construction in 1937 all of the required preliminary investigations were finished.

Geology of the site

At the damsite it was found that the native rock underlying the river bed was limestone which changed to dolomite under each of the river banks. The bedding planes in general were almost vertical. Many of the joints and bedding planes were filled with clay to a considerable depth; soluble seams and solution channels were found throughout the area; these were sometimes clay-filled, at other times open. The reservoir rim was found to be adequate, probably requiring no treatment except a short section located about four miles from the development where a small amount of remedial work may be required.

On the right bank of the river and immediately downstream from the dam there was an important dolomite quarry fully developed and in operation. This quarry was purchased not only for its value for site protection and construction space but also with the idea of using it for the production of aggregates for the concrete structures of the hydroelectric development. Prior to the time in 1937 that it was finally decided to proceed with construction, this quarry had only manufactured a coarse aggregate mainly used for road building and railroad ballast purposes. An investigation by the project engineers showed that satisfactory natural sand for making concrete could not be obtained within a radius of 200 miles of the damsite with the result that an investigation was made which showed that good stone-sand could be produced at the quarry. The excellent

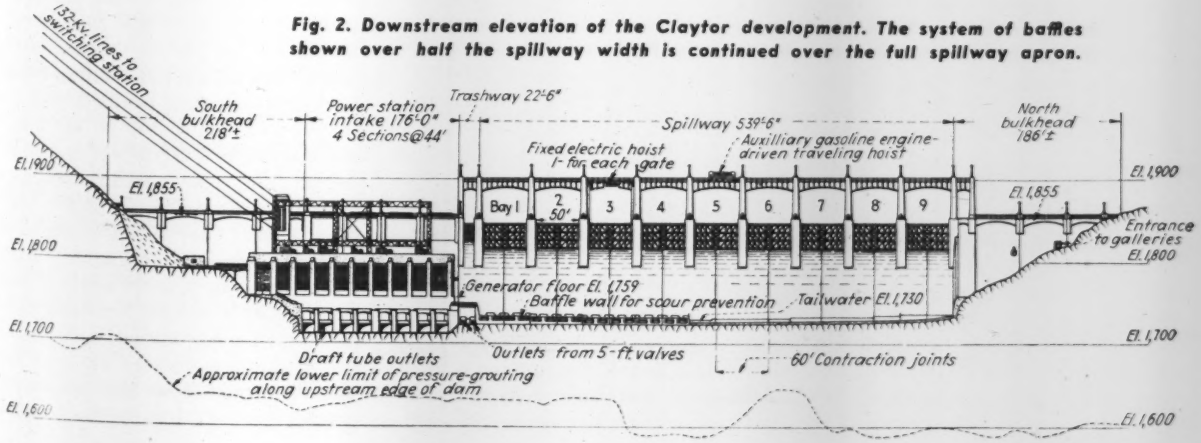


Fig. 2. Downstream elevation of the Claytor development. The system of baffles shown over half the spillway width is continued over the full spillway apron.

results that were obtained with a large saving in the cost of concrete were reported in *Proceedings*, American Concrete Institute, January, 1940.

Because the working of the quarry face on the south side of the river before its acquisition had been allowed to proceed to a location within a short distance downstream from the site finally selected for the dam and also because a wider bank was available on the north side of the river, it was decided to locate the major portion of the construction plant on the north or opposite bank. A detailed description of the construction plant was published in *Construction Methods*, April, 1939.

Grouting the rock foundation

Because of the geologic formation of the rock foundation it was important that a tight grout cutoff be constructed beneath the concrete structures of the dam and continued into each abutment.

Immediately upstream from the upstream face 3-in.-dia. holes were drilled into the foundation, first on 12-ft. centers, then on 9, 6, and 3-ft. centers. These holes, which extended to 200 ft. in depth, were pumped full of grout to refusal at 200 lb. per sq. in. On account of the nature of the foundation several areas were encountered where intermediate holes had to be drilled and grouted at less than 3-ft. centers. After adjacent grouting was completed pressure relief holes 3-in. in diameter and 45 ft. deep were drilled from the lower drainage gallery downstream from the cutoff line, to relieve any uplift pressure beneath the structure that might be created by leakage through the grouted cutoff.

When the foundation area of the dam was exposed, some sections were found to be crossed both longitudinally and transversely with clay-filled channels and seams, and exploratory drilling also revealed clay-filled pockets and joints at a relatively shallow depth. Wherever possible the clay was cleaned out by direct attack, that is, by excavating the overlying rock. In most places where this was impracticable, removal of the clay was effected with water jets or by cross-washing between drill holes. After the clay had been removed the openings were backfilled with concrete or grouted under pressure. Some sections of the dam foundation had to be consolidated by grouting in order to increase the bearing strength. These were grouted through 2-in.-dia. holes, 30 ft. deep, spaced on 10-ft. centers each way. Frequently, blanket grouting had to be performed in areas upstream from the face of the dam before the deep cutoff curtain could be satisfactorily drilled and grouted.

Naturally the acceptance of grout by the rock foundation was high, and in the case of the north bank the cutoff area averaged as much as 1.88 cu. ft. of cementitious material per linear foot of 3-in. hole drilled.

Rock fines in the grout

In order that the cost of grouting would be held to a minimum, experiments were conducted combining cement with the minus-100 rock fines produced at the quarry. It was found that a very satisfactory grout could be had by using one part cement to two parts minus-100 rock fines, with a resultant appreciable saving in the cost of injected material. No difficulties were experienced in pumping this material and according to the contractor, pump maintenance costs were not increased.

The south bank of the river was found fairly satisfactory, with acceptable groundwater levels existing close to the dam. However, investigation revealed a nearly vertical, clay-filled solution channel about 12 ft. wide parallel to the river and located about 100 ft. south of the dam. Detailed examination showed it to extend to a depth of about 30 ft. below the river bed. It was sealed by excavating the clay and building a concrete plug across the solution channel and extending it up above the reservoir level at El. 1,846.

The north bank of the river required considerable work before it would properly serve as a part of the dam. Here the rock in places was so badly folded and twisted that it was difficult to distinguish its geologic structure. The groundwater gradient was very flat, being only 30 ft. above the river elevation at a distance of 300 ft. back from the bank. Groundwater elevations consistently above the proposed reservoir elevation were not found until about 600 ft. back from the north end of the dam. Exploratory spud holes revealed the underlying rock to exist in a pinnacled form and the depth or thickness of clay overburden to vary from nothing to 70 ft. Permeability tests of the clay overburden showed it to be rather unsatisfactory and there was uncertainty regarding the results that might be obtained by chemically grouting the clay; hence it was decided to construct a cutoff through the clay overburden.

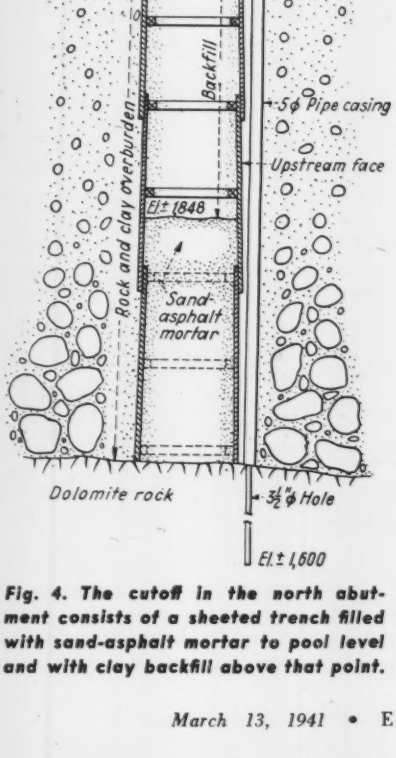


Fig. 4. The cutoff in the north abutment consists of a sheeted trench filled with sand-asphalt mortar to pool level and with clay backfill above that point.

Sand-asphalt cutoff

A timber sheeted trench about 325 ft. long was excavated down to rock and a cutoff diaphragm of sand-asphalt mortar was then constructed, as shown in Fig. 4. The conditions this excavation revealed further demonstrated the futility of trying to determine the depth of rock by spud holes, even though the spud holes were located as close as 50 ft. apart. Actual excavation, however, revealed the deepest rock not at 70 ft. but instead at as much as 150 ft. below the surface. Before deciding on sand-asphalt mortar, consideration was given to constructing the cutoff diaphragm by one of the following methods: poured concrete; clay, excavated from the trench, screened and replaced with proper consolidation; or by using an asphaltic material. The latter was finally adopted because it would have the permanent tightness of poured concrete, the

flexibility of clay and would cost less to place than either of them. Tests made in shallow parts of the trench indicated that a satisfactory bond could be obtained between the rock and the sand-asphaltic mortar. The mixture used, as proportioned on a weight basis, contained 88 percent sand and 12 percent asphalt. The sand and asphalt were separately preheated to the appropriate initial temperature, and then the combined mixture was again heated at the mixing plant to about 400 deg. F. and finally deposited in the cutoff trench at a temperature of not less than 250 deg. F. The fluid mass, as expected, spread into open seams, filled all spaces back of the sheeting, was forced along the horizontal passages and up solidly against the rock roof and generally accomplished a complete sealing of the surrounding rock.

Beneath the cutoff trench the rock foundation was grouted in a manner similar to the work performed under the main concrete structures. As previously mentioned a large amount of grout had to be injected in the north cutoff section. Quite naturally this meant that most of the grout holes as finally finished were very closely spaced.

Concreting procedure

All of the coarse and fine aggregate required for the concrete structures was manufactured from the dolomitic limestone available at the site of the development. The coarse aggregate used was graded in a series of stone or particle sizes varying from a ⅜-in. minimum to a 3-in. maximum. The fine aggregate was crushed so that all of it passed a No. 8 sieve, and 15 percent of it passed a No. 100 sieve. Aggregates were graded so that the quantity of each successively smaller size was about an equal percentage of the preceding size. In the crushing cycle adopted the material between the No. 8 and ⅜-in. size sieves was screened and recrushed to make the sand.

In the light of the meager data available on the use of crushed stone and stone sand, considerable investigation on the part of the project engineers was necessary in order that proper control would always be had over the manufacture of the aggregate, and the preparation, placing and curing of the concrete. To help in this, an adequately equipped concrete laboratory was provided at the site. Here the special investigations and routine testing were carried out. Also, additional test specimens were made to determine the freezing and thawing resistance of the concrete, especially as it was affected by the use of the minus-100 rock fines. In this connection a workability machine was developed with the help of which some very significant data on the improvements that are possible in the finished concrete by proper use of the minus-100 rock fines were obtained (A.S.T.M. Bulletin, August, 1939).

About 250,000 cu. yd. of concrete were placed in the dam, powerhouse and appurtenant structures. For the interior mass concrete of the dam 0.85 bbl. of cement per cubic yard with a maximum water-cement ratio by weight of 0.75 was used. For the exterior concrete facing the mix contained 1.2 barrels of cement per cubic yard and had a maximum water-cement ratio by weight of 0.57. The mass concrete mix averaged about 3,200 lb. per sq. in. compressive strength; and the higher cement content face mix averaged about 4,800 lb. per sq. in., at 28 days.

The upper part of one of the south abutment blocks was constructed using the mass concrete mix of 0.85 bbl. per cubic yard without the higher cement content facing, so that the relative weathering resistances of the two mixes could be compared as time goes on. The average slump of the mix used for the mass concrete was two inches and for the facing concrete three inches.

When operations were first started all concrete was mixed for a period of two minutes, but when later investigations showed that this could be cut to one and one-half minutes this procedure was adopted.

All of the concrete was placed with the aid of air-driven internal vibrators, operating at a frequency of 8,000 cycles per minute. Two sizes of vibrators were used; 3-in. for the interior mass concrete pours, and 1½-in. adjacent to the forms at the up and downstream faces of the dam. The latter was used also for the thin, heavily reinforced sections, such as intake piers, and walls.

Joints in the concrete

Referring to Fig. 2, which is a downstream elevation of the project, it will be noted that transverse contraction joints were spaced 60 ft. apart for the spillway section and 44 ft. apart for the intake bulkhead section. Although not indicated, most of the abutment sections were 54 ft. in length between joints. At the start of the job some difficulty was encountered with longitudinal cracking of the blocks parallel to the axis of the dam. In an attempt to eliminate this, various types of longitudinal construction joints were introduced in the lower series of lifts and the length of pours upstream and downstream thereby decreased. Reinforcing steel

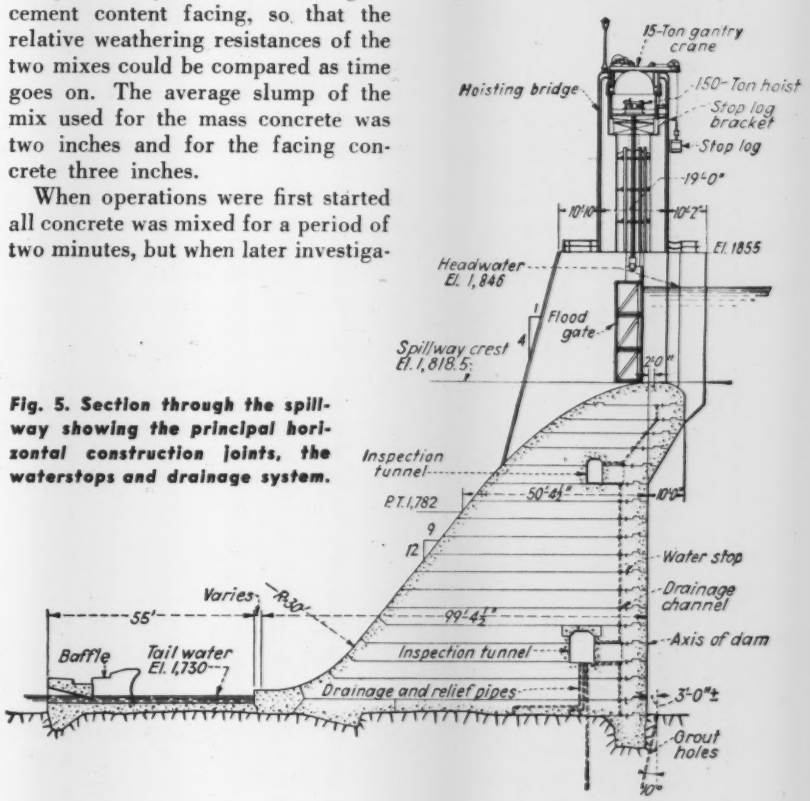


Fig. 5. Section through the spillway showing the principal horizontal construction joints, the waterstops and drainage system.

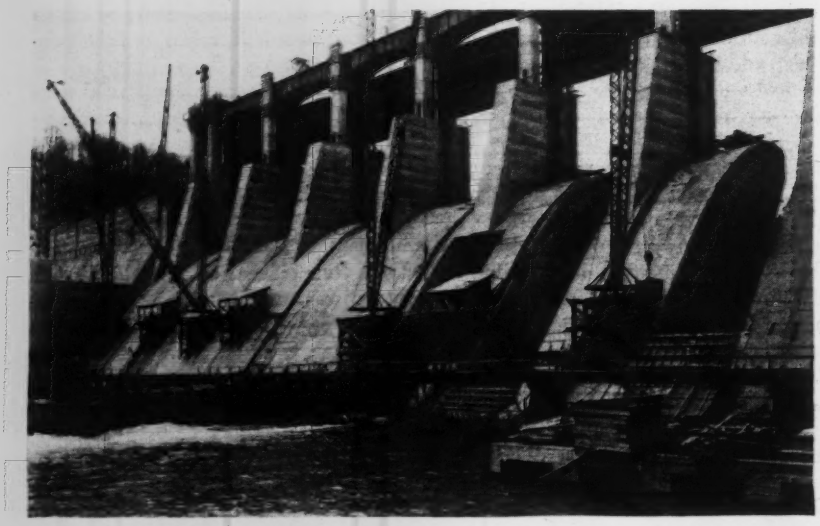


Fig. 6. Placing concrete in the passages left through the dam to handle the river during the final construction stage. After the two openings at the right had been filled with concrete to a height that would not be overtopped except during an unusual flood the upstream end of the opening at the left was closed with a steel gate which remained in place after concreting had been completed. The reservoir was allowed to build up during final stages of the closure operation.

was used in varying amounts above and below the construction joints in an endeavor to prevent cracks from extending up through the structure in prolongation of the joints when they opened. This expedient, however, was not entirely successful. In the latter part of the work satisfactory results were obtained without the use of longitudinal construction joints, that is, the successive lifts were poured monolithically for their full widths of 60, 44, 54 ft., etc., and their full transverse length of 103 ft. maximum. The height of each of the first four lifts above rock foundation was limited to 2½ ft., with subsequent lifts poured to a height of 5 ft. For a majority of the work two days were allowed to elapse between the 2½-ft. lifts and four days between the 5-ft. lifts. Whenever any of the blocks were poured to a certain height and then further lifts were delayed for several weeks, work on the subsequent lifts was handled in a manner similar to when work was started on the rock foundation.

After the job had been in progress a short time it became apparent that curing and form-removal as practiced in the average concreting procedure was not satisfactory. Therefore a separate group to inspect the curing work and plan the form removal was set up. Subsequently, Carlson resistance thermometers were embedded in the various lifts and temperatures were measured. Then a carefully planned schedule of form removal was arranged and adhered to with the result that a most gratifying improvement in quality was achieved.

Design of the dam

All of the concrete structures making up the dam were designed as gravity-type units assuming the weight of concrete at 154 pounds per cu. ft. Actually the produced concrete weighed on an average about 157 lb. Uplift was estimated on a somewhat elaborate basis, involving assumption as to an irregular gradient between head and tailwater, partial efficacy of pressure relief drains and varying proportion of area of base subject to uplift, with the ultimate result of giving about the equivalent of an applied pressure on 50 percent of the base and an intensity varying as a straight line between maximum headwater pressure at the heel and maximum tailwater pressure at the toe of the dam. This is more conservative than usual normal design assumptions. An ice pressure of 10,000 lb. per lin. ft. of dam was assumed, applied one foot below the maximum elevation of the reservoir. The structures were so proportioned for stability that under any condition of loading the resultant of the forces was always within the middle third of any horizontal plane through the structures.

An attempt was made to check development of water pressure within the concrete structures by placing waterstops in all of the horizontal construction joints and connecting them to the vertical waterstops in the contraction joints. Immediately downstream from the waterstops in the horizontal construction joints, drains were provided and connected by vertical pipes to the drainage tunnels. All of this is clearly shown in Fig. 5, a cross-sectional drawing of the spillway section. The transverse contraction joints were provided with suitable piping so that if it should be required in the future they can be sealed by grouting. Observations in the drainage tunnels thus far indicate a minimum of leakage; this can probably be attributed to both the use of the waterstops and the good bond obtained between successive lifts of the concrete structures.

Provision for flood flows

During the 35-yr. period for which flow records are available, the greatest flood flow of the New River at the damsite was 165,000 cfs. With this record, the spillway section of the dam has been provided with nine modified Broome-type gates, each 50 ft. long by 28½ ft. high, sufficient to handle a flood flow of 246,000 cfs. with no rise of headwater elevation. Each of the spillway gates is operated by a 150-ton electrically driven hoist centrally located between the bridge girders above each of the gates. Pushbutton control is provided both from the bridge over the spillway and from inside each of the gate hoist mechanism housings. For emergency operation of the spillway gates two portable gasoline engines were provided which can be readily connected to take the place of any electric hoist motor which might fail. Electric side-seal heaters were installed on all of the gates. A bubbler system was provided in front of each gate employing compressed air to prevent the formation of ice immediately upstream from the gates. In addition, five of the gates were equipped with bottom seal heaters so that there would be no danger of not being able to operate the spillway gates during freezing weather.

Nine stop-log sections sufficient for closing one spillway gate opening were constructed and are stored on brackets built out from the upstream bridge girder. One section is stored

over each gate and a 15-ton gantry crane operates over the spillway bridge to set them into position. This same gantry crane carries one of the gasoline engine drives for emergency operation of the spillway gates.

Downstream from the Claytor development river conditions are such that under normal conditions the water is only about 6 ft. deep immediately below the dam. Because of this limited depth of tailwater, together with the numerous clay-filled seams existing in the foundation rock, and the high rate of discharge per linear foot of spillway determined by physical limitations, it was feared that during flood releases raveling of the rock foundation would occur downstream from the toe of the dam. An extensive series of model tests was carried out at the hydraulic laboratory of Cornell University, and after investigating the results obtained with all familiar types of baffle walls the type shown in Fig. 5 was finally decided upon. The model gave excellent results in the laboratory and it is expected that the design will perform equally well in actual practice. As a further precaution, the electrical operation of the spillway gates is so arranged that the gates can be lifted only one foot at a time. Electrically operated indicators are provided at each pushbutton control station to guide the operating personnel in carrying out a prescribed schedule of gate operation designed so that when any gate is opened there always will be a depth of tailwater downstream from that gate sufficient to check the velocity before it could attack the rock. This will minimize the danger of erosion which might have resulted from operating the gates in a non-coordinated manner.

Organization

Design and supervision were carried out by the American Gas & Electric Service Corp. with the authors in general charge and with H. S. Slocum as construction engineer on the job. F. W. Scheidenhelm, New York, was consulting engineer. The general contractor was Rinehart & Dennis Co., Charlottesville, Va., with H. D. Faulconer as general superintendent and R. C. Perkins as superintendent on the job.

Articles on the electrical features of the Claytor development are being published in current issues of *Electrical World*.

Frameless Cellular Steel Design For Hotel of Unique Layout

***Contents in Brief**—Welded units of steel sheets in double wall sections used for walls and roof of frameless one-story hotel arranged in radiating wings from a central domed lobby. Panel boxes of special size used for windows and doors. Dome sections set by revolving jig and welded directly into position.*

USING a frameless cellular steel type of construction, the R. G. LeTourneau organization has completed a single-story hotel of unique design at Toccoa, Ga. The structure consists of a lobby 100 ft. in diameter from which radiate seven wings at 45-deg. spacing, with the eighth space occupied by the front entrance to the structure. Use of the cellular steel units for the walls, flat roof and dome of this structure follows the original installation of this type of construction used last year for a 130x284-ft. addition to the LeTourneau plant at Peoria, which was described in *ENR*, Oct. 10, 1940.

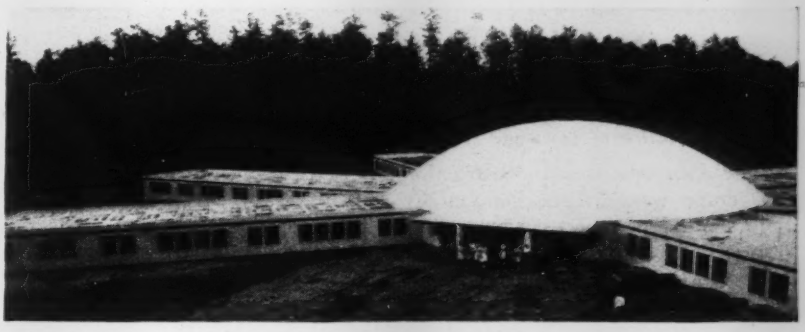
The structure is located on a site overlooking Lake Louise near Toccoa in northeastern Georgia. The area, as indicated in the illustration, was covered by pine forest which was removed and the site leveled by usual construction operations and equipment. The arrangement of the unique structure provides a central lobby, 100 ft. in diameter, which is roofed with a spherical dome from which radiate the seven 32x120-ft. wings containing the hotel rooms. These wings are spaced at 45-deg. intervals around the circular lobby and the space which would have been occupied by the eighth wing provides area for the entrance to the lobby. The arrangement provides outside windows and adequate natural light for the rooms of the hotel.

Following the grading of the site, the sewer facilities were installed, and the concrete footings and floor slabs were poured. Around the periphery of the entire building, 6-in. junior I-beams were placed in position and welded end to end. These I-beams, which were set to exact line and grade, aided in the screeding and finishing of the concrete floor. They also provided for a welded connection between the wall sections and the foundation.

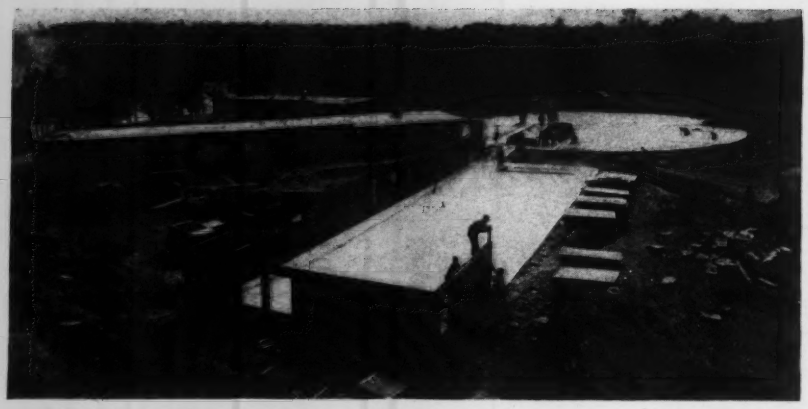
Fabrication

The entire structure is frameless and was constructed (with the exception of the dome) of rectangular panel boxes consisting of two embossed rust-resisting steel sheets spaced with steel struts which are bent to form a shallow "V" section (see article *ENR*, Oct. 10, 1940, for design drawings of panels). These struts which separate the double walls are punched to reduce the area of cross-section which helps to retard the transmission of heat between the surfaces. Panel boxes of special design were fabricated to accommodate the windows and doors.

The panel boxes are preassembled in a shop with the aid of a jig which opens like a book, and in which the



Radiating from a 100-ft.-diameter domed lobby are seven 32x120-ft. wings containing the rooms of this unique hotel.



Assembling the double-wall steel sections on the concrete base. Note the area for the central lobby in the background and the special wall sections stacked in the foreground which provide for the windows. The bottom edge of each wall section is welded to a steel I-beam imbedded in the concrete foundation.

two embossed steel sheets are placed against stops and held by electromagnets, while the struts are welded into position to form the complete panel. At the job the panels are set in position by hand (see illustration) with the outer surface of the wall placed flush with the outside edge of the I-beams imbedded in the concrete foundation. In this position the sections are welded along the bottom to the I-beam.

To make the joints between the floor and the panel wall sections vermin-proof, a 2-in. depth of concrete mixture using vermiculite was deposited between the walls, which effectively seals all openings between the wall and floor. The mineral vermiculate is an altered form of mica with much finer texture.

The dome over the circular lobby provided the feature of special construction interest on the structure. The panel boxes which are used for the dome were fabricated by the same general process as the rectangular boxes used for the walls with the exception of their shape. The inner and outer surfaces of the dome are 18 in. apart and represent two concentric spheres generated from a common center.

Erection of the dome

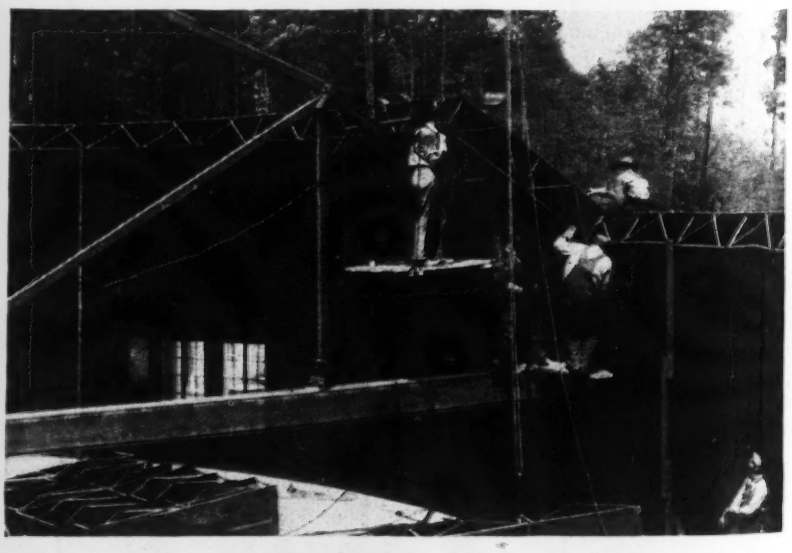
For fabrication and construction the dome was divided horizontally into seven equal zones which in turn were divided radially into a number of equal sections. Dimensions were calculated and dies made for the inner and outer sheets in each of the seven zones. Shop jigs were then made to hold the inner and outer sheets in correct position and the units were fabricated from bottom to top zone to expedite field construction.

These panel boxes were placed in position in the dome by means of a rotating arm pivoted at the center of the 100-ft.-dia. room. This assembly arm was constructed of box beams, and was designed so that they could be taken down and removed from the structure through a standard double door.

The arm of this erection jig rotated just inside the inner surface of the dome and supporting pads were provided to which the individual panel boxes for the bottom zone were fastened into position after being lifted from the ground by chain block. After the first box was placed, succeeding units were fastened to the jig and rotated into position for tack welding. This process was continued around the lower row of panel boxes until the circular section was completed. The pads for this lowest row were then removed from the jig and the next set was placed in position for the second row of panels. This removal of the first set of pads eliminated the possibility of the arm fouling on the sections already in position. Each zone of panel boxes provided sufficient rigidity to require no bracing, which greatly facilitated the use of the rotating arm as the erecting unit. The dome contains a circular ventilator opening at the center.

The walls are insulated by the filling of the space with vermiculite. The floors are covered with linoleum blocks and the interior walls decorated. One wing of the building is used for kitchen and dining room and the remaining six for guest rooms. The structure is heated by a conventional central steam heating plant.

R. G. LeTourneau is president of R. G. LeTourneau, Inc., Denn M. Burgess, general manager, and Loiell Hyler is head of the engineering department.



Forming the dome of preassembled units placed in position by revolving jig. The supporting pads on the jig were removed and moved into position for the next row of panels. The inner and outer surfaces of the dome, which are 18 in. apart, represent two concentric spheres generated from a common center.

New Specifications Issued For Defense Highway Design

***Contents in Brief**—New specifications to be applied in construction or reconstruction of roads in the defense highway network have been developed by the Public Roads Administration. They are based on standard specifications of the American Association of State Highway Officials with adjustment to meet conditions created by the movement of military equipment.*

DESIGN STANDARDS for bridges and roads adequate to meet national defense requirements have been developed by the Public Roads Administration in consultation with the War Plans Division of the General Staff of the Army. They are based on specifications of the American Association of State Highway Officials, the principal changes being those made necessary by the loads imposed by heavy tanks and by the need for greater roadway widths to permit military convoys to move over the highways without undue interference with normal traffic. The specifications for shoulder widths are the most radical departure from current practice.

For bridges and highways in rural areas the Public Roads Administration finds that a load capacity conforming to the recently revised standard H-15 live loading recommended by the American Association of State Highway Officials is adequate. These standards already are being widely used for federal-aid highway work. In urban areas use of the standard H-20 design loading is recommended as more consistent with the greater frequency of application of extremely heavy gross loads.

The Public Roads Administration states that it does not intend to enforce these standards arbitrarily on all defense projects without reference to the particular conditions surrounding each project as has been reported in some quarters. These standards are set up as a yardstick for securing a reasonable measure of the adequacy of highways to serve defense needs. Actual planning will be carried forward in cooperation with state highway departments.

New specifications

The new specifications are contained in a report entitled "Highways for National Defense" submitted to the President on Feb. 1 by John M. Carmody, administrator, Federal Works Agency (*ENR*, Feb. 20, p. 289). Those sections that depart from the standard specifications of the A.A.S.H.O. are as follows:

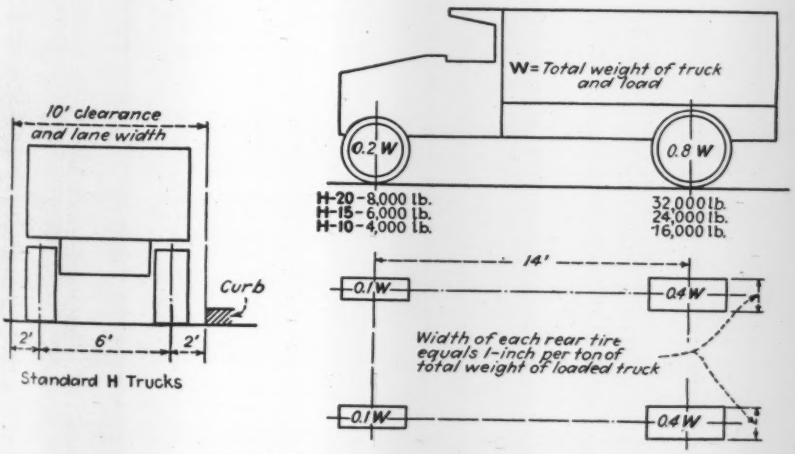


*Fig. 1. **Standard truck loadings and clearances recommended by the American Association of State Highway Officials.***

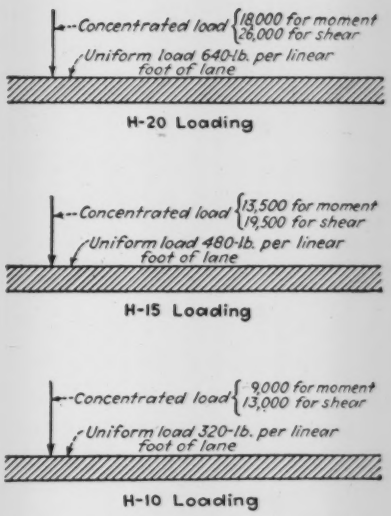

Concentrated load { 18,000 for moment 26,000 for shear
Uniform load 640-lb. per linear foot of lane
H-20 Loading
Concentrated load { 13,500 for moment 19,500 for shear
Uniform load 480-lb. per linear foot of lane
H-15 Loading
Concentrated load { 9,000 for moment 13,000 for shear
Uniform load 320-lb. per linear foot of lane
H-10 Loading

Fig. 2. Lane loadings recommended by the American Association of State Highway Officials for three classes of highways.

Within the usually assumed tolerance of overloads, bridges of design and condition capable of supporting H-15 loading will safely support the loads of all commonly used commercial vehicles in the frequency with which they are normally applied. They will also similarly support the loads of all military equipment other than heavy and medium tanks, without special control of the movement.

Regarding the support of tanks, special studies have been made from which the following conclusions can be drawn as to the effects of the specific classes of tanks shown diagrammatically in Fig. 4 on standard H-15 bridges of width equal to two or more traffic lanes:

Light (13½-ton) Tanks—Simultaneous loading of two lanes with these tanks does not produce greater than allowable stresses.

Medium (20 and 25-ton) Tanks—If the movement is controlled so that there will be no more than a single file of such tanks following approximately the center line of the bridge

and spaced at least 40 ft. apart, no greater than allowable overstresses will result. There will be no necessity to limit the speed of the tanks.

Heavy (55-ton) Tanks—Such tanks will be safely supported if the movement is controlled so that there will be no more than a single file following approximately the center line of the bridge, with separating intervals of not less than 50 ft., and with speed (on the bridge) reduced to not more than 4 m.p.h.

Horizontal clearance

Existing bridges and underpasses on 2-lane roads having a horizontal clearance of less than 18 ft. should be replaced or widened as promptly as possible.

New and widened bridges on 2-lane roads should have at least the minimum, and if feasible, the preferred horizontal clearances indicated in the following schedule of widths corresponding to various classes of present average 24-hr. traffic volume:

Present average 24-hr. traffic volume	Horizontal clearance of new or widened bridges	
Number of Vehicles	Minimum, in feet	Preferred in feet
Less than 600	24	26
600 to 1,800	26	28
More than 1,800	28	30

The minimum 2-lane bridge clearances specified are substantially adequate, and the preferred clearances completely adequate for civil traffic, including the usual small percentage of vehicles of width equal to and slightly exceeding 8 ft. They are similarly adequate for the accommodation of mobile military equipment, the maximum width of which (exclusive of the 25 and 55-ton tanks) is approximately 8½ ft.

Existing 2-lane bridges affording horizontal clearance of less than 20 ft., if not widened, should be posted as narrow bridges. Existing bridges on 3 and 4-lane roads should be replaced or widened as promptly as possible if, in horizontal clearance, they afford less than 30 and 40 ft. respectively.

New and widened bridges on 3 and 4-lane roads should have horizontal clearances of at least 40 and 52 ft. respectively, and preferably horizontal clearances between extreme lateral curbs should be 6 ft. greater than the width between the outer edges of an approach pavement or pavements designed for 12-ft. lanes. Bridges on 4-lane roads having a wide central dividing strip may be built as dual bridges, each bridge providing horizontal clearance of at least 26 and preferably 30 ft.

New and widened underpasses on 2-lane roads should have at least the minimum, and if feasible the preferred, horizontal clearances indicated in the following schedule of widths corresponding to various classes of present average 24-hr. traffic volume:

Present average 24-hr. traffic volume	Horizontal clearance of new or widened underpasses	
Number of Vehicles	Minimum, in feet	Preferred, in feet
Less than 600	30	40
600 to 1,800	30	42
More than 1,800	30	44

New and widened underpasses on 3 and 4-lane roads should have horizontal clearances at least 6 ft. and preferably 20 ft. wider than the width between outer edges of an approach pavement or pavements designed for 12-ft. lanes.

Existing bridges and underpasses having a vertical clearance of less than 12½ ft. should be heightened as promptly as possible.

New and heightened bridges and underpasses should have vertical clearance of at least 14 ft. A vertical clearance of 14 ft. is sufficient for the passage of all military equipment and substantially all vehicles of normal commercial traffic.

Existing bridges and underpasses affording vertical clearance of less than 14 ft. if not heightened, should have their minimum clearance plainly posted upon them.

Road surfaces and foundations

All road surfaces should be dustless; and all surfaces and foundations should be designed in accordance with the present practice of each state for the all-weather support of repeated applications of 9,000-lb. wheel loads on pneumatic tires, wheel load being defined as half the axle load where dual wheels are used.

Road surfaces designed in accordance with this recommended standard

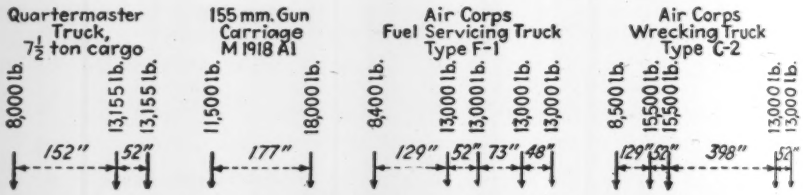


Fig. 3. Typical wheel-load concentrations of the heavier types of military equipment that will move over the defense highways.

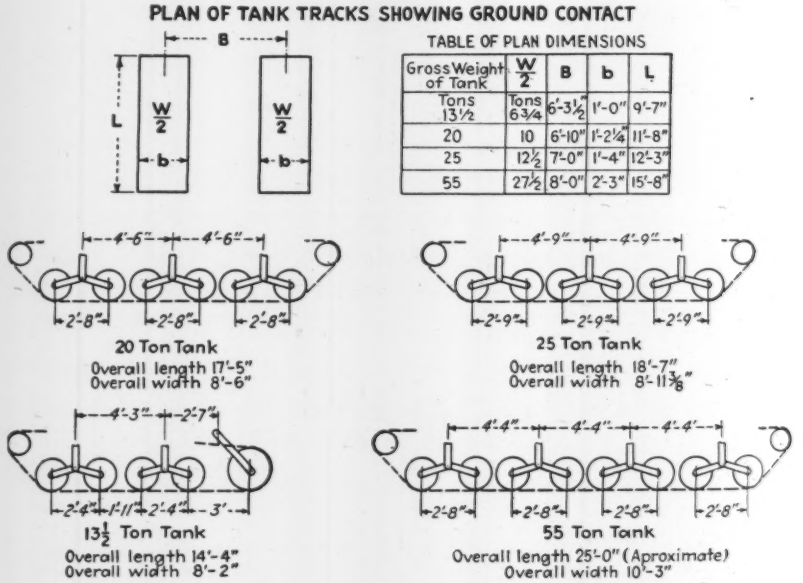

PLAN OF TANK TRACKS SHOWING GROUND CONTACT
B
L
W/2
b
TABLE OF PLAN DIMENSIONS
Gross Weight of Tank | W/2 | B | b | L
Tons 13½ | Tons 6¾ | 6'-3½" | 1'-0" | 9'-7"
20 | 10 | 6'-10" | 1'-2¼" | 11'-8"
25 | 12½ | 7'-0" | 1'-4" | 12'-3"
55 | 27½ | 8'-0" | 2'-3" | 15'-8"
20 Ton Tank
Overall length 17'-5"
Overall width 8'-6"
25 Ton Tank
Overall length 18'-7"
Overall width 8'-11⅜"
13½ Ton Tank
Overall length 14'-4"
Overall width 8'-2"
55 Ton Tank
Overall length 25'-0" (Aproximate)
Overall width 10'-3"

Fig. 4. Loading diagrams for standard tanks and spacing of the tank tracks for the four types in common use.

will be completely adequate for the accommodation of all normal civil traffic and all traffic of military vehicles, including all classes of tanks at present in use and projected.

Two-lane Pavement Width—Existing 2-lane roads having a pavement width of less than 18 ft. should be widened as promptly as possible.

Newly constructed and widened pavements on 2-lane roads should be designed according to the following schedule of widths corresponding to various classes of present average 24-hr. traffic volume:

Present average 24-hr. traffic volume Number of Vehicles	Width of new or widened pavements in feet
Less than 600	20
600 to 1,800	24
More than 1,800	24

The widths specified are substantially adequate for all normal civil traffic, including the usual percentages of wide and heavy vehicles, with range of speed from 15 to 70 mph. They are completely adequate for all anticipated military usage.

Three and Four-Lane Pavement Widths—New 3- and 4-lane pavements should be constructed to provide a width of 12 ft. per lane. Pavements so constructed will be completely adequate for all civil and military traffic. It will be unnecessary to widen any serviceable existing 3 and 4-lane pavements, the lane width of which is less than this recommended standard.

Fig. 5. Wide shoulders are needed if military convoys are to move over the nation's highways without serious interference with normal traffic. For lack of wide shoulders this convoy cut an important road down to little more than one lane.

Shoulders

On roads carrying a present average daily traffic of more than 1,800 vehicles and on less heavily traveled roads wherever feasible, it is desirable that shoulders be provided continuously on both sides of the pavement of sufficient width to permit parking. For this purpose a minimum width of 8 ft. is essential. Such provision is desirable for the accommodation of normal civil traffic; it is highly desirable on roads that are likely to be used with some frequency by military convoy movements, to prevent excessive interference with moving traffic by halted convoys unable to clear the traffic lanes.

If, on roads that are likely to be frequently used by convoys, it is not feasible to provide continuous shoulders of a width sufficient for parking, shoulders of such width should be provided on sections not less than 2,000 ft. in length at intervals of not more than four miles. Such shoulder sections should be staggered on the two sides of the road, so as to provide space for parking on one side or the other at intervals averaging not more than two miles in length. On roads carrying more than 1,800 vehicles per day this should be regarded as a minimum requirement, and exception should be considered only in mountainous locations involving the heaviest grading.

At no point should shoulders be less than 4 ft. wide.

Grades and curvature

It will be unnecessary to change the grades of any road otherwise adequate.

On roads which it is necessary for other reasons to construct or reconstruct, it is desirable to hold maximum grades (in excess of 500 ft. long) within the limits of 6 percent in mountainous locations and 4 percent in non-mountainous locations.

Grades of the recommended limiting percentages will permit operation by the majority of all passenger automobiles at speeds of 60 and 70 mph. respectively. They will permit operation by the majority of motor trucks, with reasonable carried loads, at speeds in excess of 25 mph. and by the majority of tractor-semitrailer combinations, reasonably loaded, at speeds not less than 15 mph.

It will be unnecessary to revise the curvature of any road otherwise adequate.

On roads which it is necessary for other reasons to construct or reconstruct, it is desirable to hold maximum curvature within the limits of 6 deg. in mountainous locations and 4 deg. in non-mountainous locations.

Curvature corresponding to the recommended limits, combined with maximum practicable superelevations, will permit safe operation at speeds of 60 and 70 mph. respectively. In mountainous areas where heavy grading is required, and on low-priority routes where traffic is light, maximum curvature may be increased to 14 deg. On low-priority routes in non-mountainous areas where traffic is light, maximum curvature may be increased to 10 deg.

The 1935 edition of the highway bridge specifications of the American Association of State Highway Offi cials that served as the basis for these new specifications is out of print but a 1941 edition will be off the press soon. Copies may be obtained from the association, National Press Bldg., Washington. Price $2.

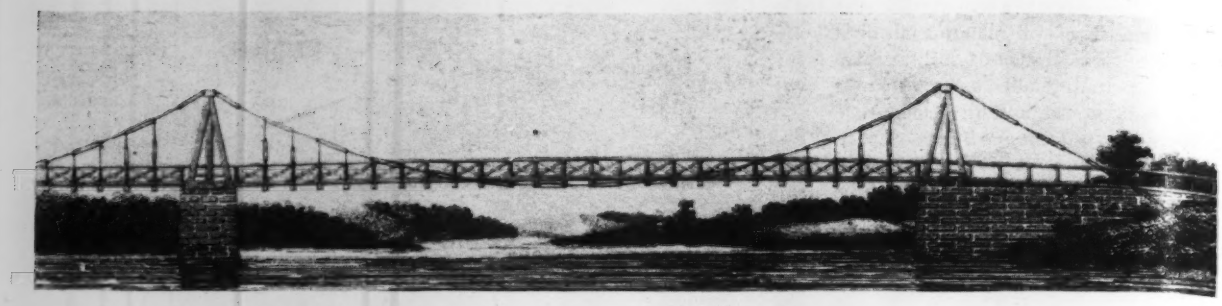


Fig. 1. Finley design of the early 1800's in which the railing first appeared as a substantial stiffening truss.

Wind Failures of Suspension Bridges
or
Evolution and Decay of the Stiffening Truss

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Columbia University, New York City

***Contents in Brief**—History was merely repeating itself when wind-induced undulations of the deck of the Tacoma Narrows Bridge built up forces that could only be relieved by collapse of the structure. Search of the records reveals numerous accidents of similar nature, attended by the selfsame oscillations. And in all cases it was lack of stiffening or rigidity that caused the difficulty. When heavy stiffening trusses were used, the oscillations ceased. In recent years, a tendency to compromise with rigidity has brought the oscillations back.*

THE BASIC IDEA of the suspension bridge—the chain or cable securely anchored at each end and supported by towers with a nearly horizontal roadway suspended from it—seems to be one of the common inheritances of mankind. Jungle people the world over have built crude suspension bridges and it has been facetiously suggested that our simian ancestors found vines and creepers such valuable aids to locomotion that they have left a lasting impression on the human race. Nevertheless the modern suspension bridge is a relatively recent invention. Its development was dependent primarily upon the availability of suitable material—strong and lasting—for the supporting element. The production of wrought iron, both in reasonable quantities and at a reasonable cost in the last years of the 18th century, made the modern suspension bridge possible. The first bridges of this type were all wrought iron chain suspensions. The introduction of the wire cable was, apparently, a French contribution and came later.

It seems clear that the father of the modern suspension bridge was an American engineer, James Finley (1762–1828), of Fayette County, Pennsylvania. Finley writes: "In the year 1801, I erected the first bridge of this construction on the turnpike between Uniontown and Greensburg, Pa., in a contract . . . to build a bridge of 70-ft. span, 12½ ft. wide, and warrant it for 50 years (all but flooring) for the consideration of $600. Nothing further of the kind was attempted for six years. The exclusive right was secured by patent in 1808."[1]

By 1810 six Finley bridges had been erected, and in a paper published in that year Finley describes his method of construction.[1] It is interesting to note that his bridge (Fig. 1) of 1810 embodies all the elements of the modern suspension. Note in particular that the side railing is developed as a strong and well-braced stiffening truss. Finley does not so describe it but he suggests the construction of the upper and lower chords of this railing in a continuous form, to give "stiffness" to the bridge, by using overlapping layers of 2½-in. plank. Clearly this gives some stiffness to a type of structure which is inherently flexible and would otherwise deflect excessively under concentrated loads. This truss makes the bridge, in effect, a long, shallow truss, far too long to support its loads without the suspension afforded by the cables, but capable of distributing a load over a sufficient length of cable to prevent excessive local deflection of the floor. While the floor construction of one notable bridge built after Finley's designs—the 240-ft. chain suspension built in 1810 over the Merrimac at Newburyport, Mass.[2]—does not seem to have involved these stiffening railings, they were added in about 1900 to permit electric cars to use the bridge.

In short, the suspension bridge, as originally conceived, was a combination structure—a suspended truss—and it is the evolution of this combination design, particularly of the truss element, that is today of particular interest.

Disaster to flexible British bridges

In the first phase in the evolution of the suspension bridge, however,

the principal effort was directed toward the improvement of cable construction. The stiffening truss was disregarded—with disastrous results. British builders, in particular, adopted the suspension principle but used relatively light "platforms" for the suspended roadway with extremely light railings that had practically no stiffening value. Apparently no difficulty was experienced with excessive lateral deflection due to wind, for the platforms were relatively stiff in a transverse direction. But they were woefully weak to vertical displacement under aerodynamic wind action. Fortunately clear descriptions by competent observers are available of the action of wind on several of these early British bridges.

The suspension bridge pioneer in Great Britain was Captain Sir Samuel Brown (1776–1852), who introduced the chain cable in the British Navy and patented, in 1817, a chain suspension bridge. Among his constructions of this type were the Union Bridge over the Tweed at Berwick, 448-ft. span, built in 1819–20, which was wrecked by wind six months after completion, his famous Chain Pier at Brighton, which suffered from like cause in 1823 and again in 1836, and the Montrose Bridge in Scotland, on which some interesting data are available.[3]

The Montrose Bridge, built in 1828–29, had a suspended roadway 412 ft. long and 26 ft. wide supported by four main chains, a pair vertically above each other on each side of the roadway. Massive masonry towers were used. The "bearers," or floorbeams, were hung from the suspenders at intervals of 5 ft., and carried longitudinal 3-in. planking covered with a transverse sheathing $1\frac{1}{2}$ in. thick. A very light iron railing of lattice type was placed on each side of the roadway, clearly to function solely as a railing.

There was a failure of one of the main chains in 1830, due to overloading of the bridge during a boat race, but the chains were repaired and all was well until Oct. 11, 1838, when the roadway was partially destroyed by a "hurricane."

An undulating motion was observed during this storm. While it was stated that "this motion was greatest at about midway between the towers and the center of the roadway," the description of the undulating motion is not clear. It resulted, however, in the collapse of a section of roadway in the center of the span.

Menai Straits Bridge

The next record is more complete. Thomas Telford (1757–1834), the great British road and bridge builder, built a major span, 550 ft. with a clearance of 100 ft. (Fig. 2) which was opened in 1826 over the Straits of Menai to the Island of Anglesay in east England on the post route from London to Holyhead and Ireland. It had four main cables of wrought iron arranged in vertical groups of four. The floor system was of timber and the railing, although 7 or 8 ft. high, had little stiffening action (Fig. 2).

The Menai Bridge was opened Jan. 30, 1826. The following month a heavy gale caused so much motion in the chains and roadway as to break several of the vertical suspender rods and many of the bearers (i.e., floorbeams) which supported the platform.

To quote: "The motion which had been anticipated was that of simple undulation flowing in waves at right angles to the length of the bridge. . . . The movement of the undulating wave, however, was oblique . . . with the general direction of the bridge. In other words, when the summit of the wave was at a given point on the windward side of the bridge, it was not opposite this point on the leeward side, but in relation to the flow of the wave, considerably behind it; the motion appearing to be generated on the windward side, and by the time it had crossed to the leeward, it had also moved forward along that on which it had commenced. . . The motion was observed to be greatest about half way between the pyramids (i.e., towers) and the center of the bridge. The wave increased in its progress from the pyramid till it attained its maximum altitude near the first quarter, and at the same instant the extreme depression was near the third quarter. The wave then gradually diminished to the center of the bridge and afterwards increased to the third quarter, when it attained its greatest height at the same time that the first quarter was most depressed. The platform and chains were equally subjected to this undulatory motion."

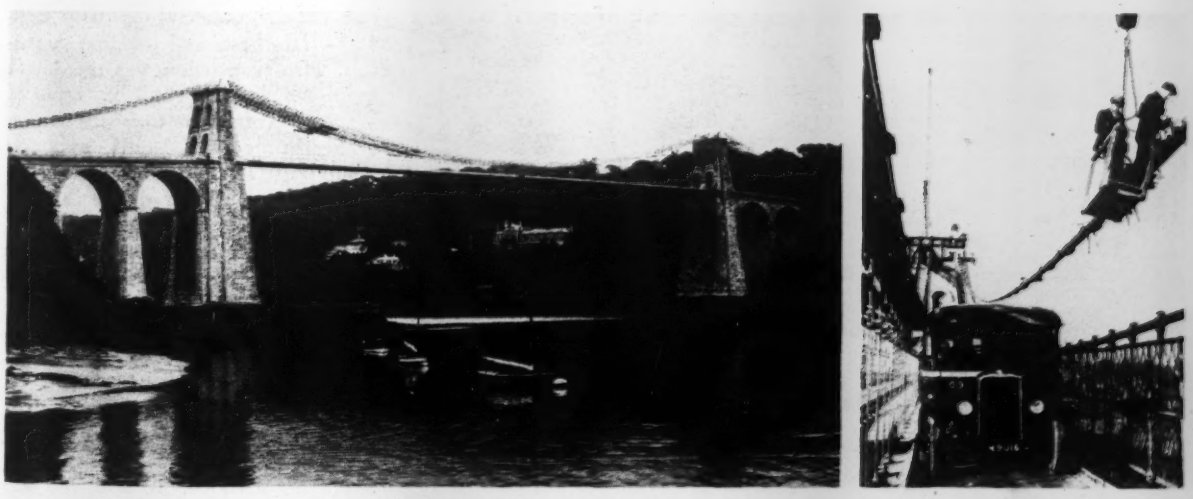

Fig. 2. Telford's Menai Straits Bridge in Wales, built in 1826, which in its early years exhibited the same type of aerodynamic instability that wrecked the Tacoma Narrows Bridge. It has no stiffening truss to speak of. View of right taken in 1939 shows workmen removing wrought iron chains in a re-cabling operation.

Fig. 3. Ellet's Wheeling Bridge of 1,100-ft. span over the Ohio River, whose destruction by wind in 1854 was preceded by the same form of undulation that wrecked the Tacoma Narrows Bridge.

Some repairs and strengthening of the floor was undertaken and, apparently, all went well until January, 1836, when a severe storm again caused trouble. The bridge keeper, who had seen the bridge through every storm, estimated that "there was little less than 16 ft. between the extreme of rise and the lowest point of depression" in the undulations which occurred at this time. No great damage was caused.

"The memorable hurricane of January, 1839," however, destroyed in part the roadways (footway still could be used) and necessitated major repairs. The breaks were approximately at the quarter points of the span, i.e., at the points of maximum movement.

If we recall the Tacoma failure, the above description of the behavior of the wind action at Menai affords a remarkable parallel. In fact these long forgotten difficulties with early suspension bridges clearly show that while, to modern engineers, the gyrations of the Tacoma bridge constituted something entirely new and strange, they were not new—they had simply been forgotten. The most remarkable parallel to the Tacoma failure, however, is to be found in that of the Wheeling, W. Va., suspension bridge in 1854.

While British builders stuck to chain construction, American workers after Finley adopted wire cables. The Wheeling bridge was of wire cable construction.

This type of cable was, apparently, a French contribution. A Colonel Dufour built a wire bridge in 1823, but it was Louis J. Vicat (1786-1861), of cement fame, who, in his Argental Bridge of 1830, developed a method of building up the wire cables in place and strand by strand, the "cable spinning" that has been followed ever since. Another French engineer, Joseph Chally (1795-1861), following this method, built a record-breaking span of 810 ft. at Fribourg, Switzerland, in 1834. Fourteen years later the leadership passed to America, where it has remained.

Fig. 4. Roebling's 825-ft. railroad and highway span over the Niagara gorge not only had a deep stiffening truss but its deck was anchored to the ground by diagonal "storm cables".

The Wheeling Bridge

The first notable American wire suspension bridge builder was Colonel Charles Ellet, Jr. (1811-1862), an Army officer of note and one of our outstanding early engineers. His first, and the first American, wire suspension was the 358-ft. span which, in 1841-42, replaced Wernwag's famous *Colossus* timber arch (burned in 1838) over the Schuylkill at Fairmount, Philadelphia. Later, after building a footbridge at Niagara, he abandoned further attempts because of difficulties between the contractor and the directors of the company, but he built, in 1847-48, a record-breaking suspension over the Ohio at Wheeling, W. Va. This bridge had a span of 1,010 ft., a 17-ft. roadway, and a 3½-ft. sidewalk. Ten large and two small cables were used, the larger requiring 550 No. 10 iron wires—about the same size still used in cable construction.

In 1854 the floor of this bridge was

completely destroyed by what was described as a hurricane. In the *Wheeling Intelligencer* of May 18, 1854[5] an eye-witness describes the failure of this bridge. He records that on the afternoon of May 17 he walked up the river bank toward the bridge intending to cross, as he had many times before, to experience the cool breeze and, as he rather naïvely remarks, "enjoy the pleasure of the undulating motion of the bridge." There was a strong wind blowing, however, and he soon discovered that the structure was behaving in a most peculiar manner.

"For a few minutes we watched it," he wrote, "with breathless anxiety, lunging like a ship in a storm. At one time it rose to nearly the height of the towers, then fell, and twisted and writhed and was dashed almost bottom upward. At least there seemed to be a determined twist along the entire span, about one half the flooring being nearly reversed, and down went the immense structure from its dizzy height to the stream below, with an appalling crash and roar."

Here again is a clear description of the identical mode or form of undulation which wrecked the Tacoma span.

It seems extraordinary that, with these and other records of wind difficulties, nothing constructive in the way of an analysis of wind action developed. The answer is that the stiffening truss again came into its own and with an adequate stiffening truss undulations due to wind were no longer important.

Rebirth of stiffening truss

In short, it appears that most of Finley's successors concentrated their efforts on cable building and ignored, or lost sight of, the wind-resisting function of the stiffening truss. A heavy truss was not necessary to distribute wagon loads and Finley's idea went into the discard with fatal results. But by 1850 bridge loads were on the increase, the railroad was being rapidly developed, and railroad loads being heavy and suspension bridges flexible, the necessity for substantial trusses to distribute live loads became obvious. This is the point where John Roebling (1806-1869) steps into the picture.

At the same time that Ellet's light Wheeling bridge was wrecked, Roebling's Niagara suspension with a very adequate stiffening truss was nearing completion. Roebling's first bridges were canal suspensions and carried wooden troughs. Although failure was freely predicted, these works gave admirable service and one—with trunk replaced by timber floor—still serves as a toll bridge over the Delaware at Lackawaxen.

Underwood & Underwood photo

Fig. 5. When Roebling built the Brooklyn Bridge, shipping clearances prohibited the use of storm cables beneath the deck, so he inclined them from the towers.

When Ellet withdrew from the Niagara work, Roebling, whose earlier bid had been unsuccessful, was given the contract. The Niagara suspension (Fig. 4), completed in 1855 with a span of 825 ft. and a clearance of 250 ft., was a most notable work. Its upper deck carried the railroad track, its lower deck was for horse-drawn vehicles. These decks were joined by substantial supports thus forming an effective stiffening truss.

Sway cables at Niagara

Apparently the use of the truss was not the result of rational design but of intuition and the necessity for supporting traffic on two levels. One of the first to offer a rational approximation to the design of the stiffening truss was the famous Scotch engineer-mathematician, W. J. M. Rankine (1820-1872), but it is said that his theory arrived in this country too late to affect the design even of John Roebling's last bridge, the Brooklyn Bridge of 1883. It is also clear that Roebling did not rely fully on the Niagara truss to prevent distortion by wind. He securely anchored the truss by cables leading down to the sides of the gorge (Fig. 4) and, although frequent repairs were necessary and the floor system was replaced in iron in 1881 and the towers in 1886, this bridge served successfully until it was replaced by Buck's steel arch of 550-ft. span in 1899.

Before turning to Roebling's next great work—the Brooklyn Bridge—two other Niagara suspensions deserve brief note. A pioneer bridge had been built at Lewiston, at the lower end of the gorge, in 1850-51. Guys, such as Roebling employed, had been used to hold this bridge against wind action. An ice jam in the gorge, however, led to these being temporarily removed in 1864, and the structure failed in a windstorm a few weeks later. Even stiffened railings of the Finley type failed to save the floor of this bridge but, unfortunately, details of the "oscillations" reported to have caused failure are not available.[6]

Another wind failure at Niagara was that of the Niagara-Clifton suspension. This bridge, at the mouth of the gorge just below the Falls, built 1867-69 by a Canadian engineer, Samuel Keefer, had the notable span of 1,260-ft. centers of towers, and was originally only 10 ft. wide. Additional cables were added in 1888 and it was widened to 17 ft. Within a month (Jan. 10, 1889) it failed in a windstorm that is said to have reached 74 mph. The failure occurred at night, between 2 and 3 A.M., and the mode of oscillation is thus unknown. A Dr. J. M. Hodge of Niagara Falls did, however, cross this bridge about midnight. It was a peril-

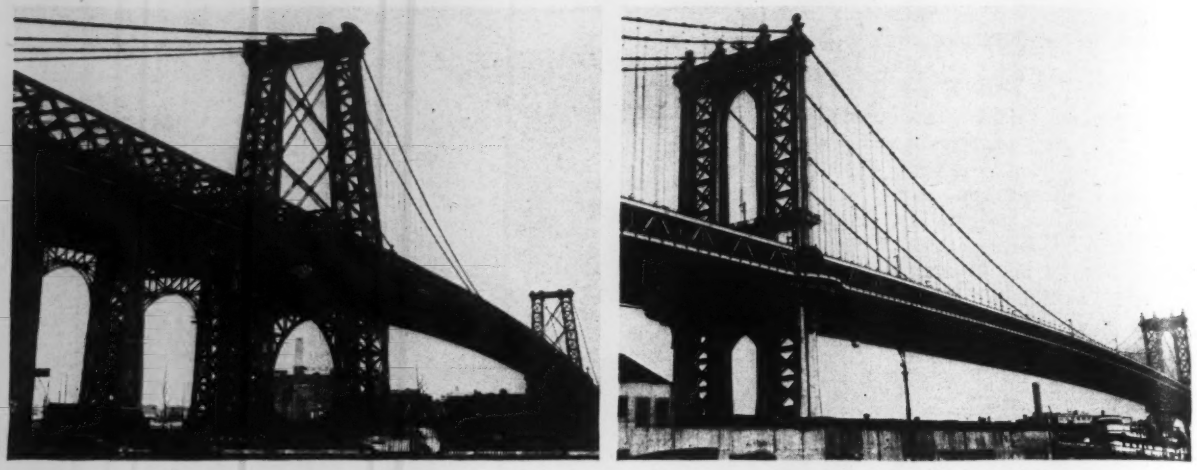

Fig. 6. First bridges that took advantage of the new theory of stiffening truss action were the Williamsburg (left) in 1903 and Manhattan (right) in 1909, both neighbors of the Brooklyn Bridge. The Manhattan, designed by Moisseiff's deflection theory, has been called the first truly modern suspension bridge.

ous trip, for he records: "The bridge rocked like a boat in a heavy sea and at times it seemed to tip up almost on its very edge." (It was repaired in four months, replaced in 1898 by Buck's famous 840-ft. trussed arch, which was carried out by ice in 1938 and is now in the course of being superseded by a 950-ft. arch of hingeless type.)

Brooklyn and the East River bridges

Roebling's greatest work, Brooklyn Bridge (Fig. 5), begun in 1869 and completed by his son, Washington A. Roebling, was one of the modern marvels of its day. Its span of 1,595 ft. was for a time the greatest in the world—the Firth of Forth cantilever of 1890, with two spans of 1,710 ft. took over this record. Its floor system, with tracks at two levels, resulted again in a substantial stiffening truss and the bridge is still in service under relatively heavy although limited loads today.

One of the characteristics of this bridge is the use of auxiliary inclined cables in addition to the usual vertical hangers or suspenders. Roebling referred to this type of stay as "storm-cables."[7] Where clearance requirements did not permit, as they had at Niagara, stays below the floor, Roebling apparently conceived the idea of reversing their position and using stays from the tower tops. That is was an effective means of preventing possible undulation in the floor is indicated by the fact that similar diagonal stays are suggested today as a means of reinforcing some of the more recent light floor bridges. These stays, however, are stressed by live load, their adjustment becomes difficult and they are not needed if a substantial stiffening truss is used, so that they were abandoned in later bridges.

Following Roebling's work, the theory of the stiffening truss—the design of the suspension bridge as combination structure—was more fully developed. With the construction of the Williamsburg Bridge in 1903, the greatly increased loads of suburban cars and the new theory of stiffening truss action led to the building of the heaviest, clumsiest suspension which ever has been or probably ever will be erected. Recalling the graceful lines of the Brooklyn Bridge, the clumsy towers, the huge stiffening truss, 40 ft. deep, and the use of vertical suspenders only, make Williamsburg look like an elephant in comparison with a giraffe.

The Manhattan Bridge, built at a location between these two, in 1909, set a more modern note. The approximate theory of the Williamsburg gave way to a more exact analysis, the "Deflection Theory," introduced by Leon S. Moisseiff. This theory took into account the combined action of cables and truss in deflection, and this structure has been called the first truly modern suspension. It was followed by such notable works as the Philadelphia-Camden of 1926, the Detroit River and other bridges.

Suspension spans outside New York were seldom designed for heavy suburban trains. The stiffening truss was thus relatively light, although it was still adequate to prevent the forgotten undulations which had plagued the earlier builders, and with which Roebling took no chances. But traffic requirements were again changing. The motor era required no such heavy construction, and funds became available for ferry replacements by long span, relatively light constructions.

Modern decline of stiffening truss

When the George Washington Bridge was built in 1931, it was designed with a stiffening truss having a highway on the upper floor and tracks for suburban trains to be later carried on a lower deck. It was found that there was no need to build the stiffening truss, inasmuch as the train service was to be added later, and this truss was not needed to distribute the relatively small live loads of motor and bus traffic. In fact, the cables were so stiff—the dead weight of such long span bridges is so great—that a loaded bus produces about the same deflection as a fly walking along a rope. The George Washington Bridge was thus the first modern bridge to be built without a stiffening truss—and it is so heavy that it needs none.

Following this experience, bridge engineers began to design narrow, light, highway suspensions with relatively shallow girders instead of stiffening trusses. As far as the distribution of vertical loads was concerned, these girders did the trick. It was, of course, true that a side wind pres-

sure would produce relatively large lateral deflections because these long, narrow, shallow girders gave relatively less stiffness to the trussing, which they formed with the floor, against side deflections. The problem, however, was considered as a matter of static loading, and side deflections of 8 or 10 or more feet were expected.

A "new" phenomenon

In the summer of 1938, however, a supposedly "new" phenomenon was observed at the Golden Gate Bridge —longest in the world, 4,200 ft. A lateral wind, estimated at 78 m.p.h., caused a side deflection estimated at 8 ft., but it also caused a wave, or undulation, to pass over the floor sufficient in amount to permit the double suspender cables to slap against each other. This was recognized as an aerodynamic phenomenon, but a stiffening truss was used for this bridge and there appeared to be no reason for expecting such movements to reach more than annoying proportions.

Nevertheless, the same phenomenon occurred at the Thousand Islands suspension, completed in 1938, and at the Deer Isle Bridge in Maine. Various devices were used, and apparently with success, to reduce these undulations. Diagonal cables between the main cable and floor, and horizontal friction dampeners at the ends where the floor joins the towers, were employed.

In fact, the Whitestone Bridge—widely acclaimed for its slenderness and grace—the "streamlined" bridge —also misbehaved. The long, light, flexible floor showed a desire to undulate, and it has not only been braced but also most carefully watched.

The lesson is plain

It is, of course, possible, even probable, that in the Tacoma bridge certain peculiarities of construction—the form of floor, the natural periods of the floor, cables and even towers —led to a magnification of the oscillations and thus to failure. But it seems clear that bridges of widely differing floor and cable construction have experienced the same mode of oscillation. History, in short, has been repeating itself, although this fact has, apparently, not been known even to engineers who have made a specialty of this type of construction. This time the problem of preventing undulations in suspension bridges will undoubtedly be solved.

Increased stiffness against wind action is clearly essential. It can be provided through the use of heavier floors (i.e., increased dead load) or with a stiffening truss; nevertheless a more economical plan may be developed. Guy cables of the Roebling type can be used to reinforce existing bridges, yet opinion is not unanimous in accepting such cables, in new projects, as a fully satisfactory solution of the problem. But as further study clarifies the basic elements in the aerodynamic problem involved, the bridge engineer will undoubtedly find a successful and economical solution of the problem. The static theory of suspension bridge design has been most fully developed, particularly under the leadership of the consulting engineer of the Tacoma bridge, and we may rest assured that the modern structural engineer will meet and solve the new dynamic problem which changing loads and economic considerations have again brought to the fore in suspension bridge design.

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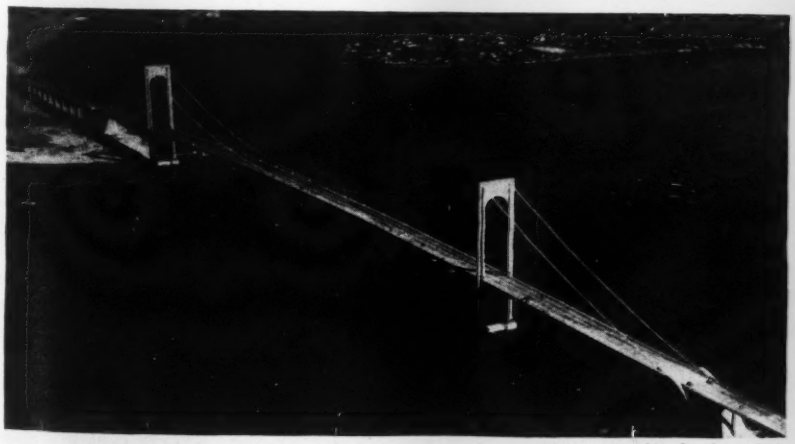


McLaughlin Air Service photo

Fig. 7. New York's Whitestone Bridge of 1939, noted for its slender, light floor and plate girder stiffening "trusses", exhibited a tendency to undulate which has been checked by brakes and sway cables.

Well Water in Indianapolis

About 45 million gallons of water a day is being pumped from wells in the Indianapolis area, of which only a little over 1 mgd is taken from wells of the Indianapolis Water Co. Most of the water—about 39 mgd—is pumped within three miles of Monument Circle, in the center of the city.

This information was obtained in the course of an investigation of the groundwater resources of the area made by C. L. McGuinness, of the U. S. Geological Survey. It is reported that about 60 per cent of the water is obtained from beds of sand and gravel of the glacial drift overlying the bedrock, and that the balance comes from limestone underlying the drift.

It was also revealed that during the summer some 4½ mgd of water is pumped for air conditioning uses. Considerable decline in water level has occurred in the downtown district, where large quantities of water have been pumped in recent years from gravel wells for air conditioning.

Mud Mountain Dam Redesigned As Rockfill with Earth Core

Contents in Brief—*Contract for a rolled fill dam with the record height of 425 ft. for flood control on the White River in Washington was let before it was discovered that the material available, under weather conditions prevailing in this location during most of the year, was "to plastic" for constructing a satisfactory dam of this type. Redesign provides for a central rolled fill core flanked by supporting zones of dumped rock. Precipitous canyon walls give the dam the anomalous dimensions of 1,600-ft. base width and 700-ft. crest length.*

Characteristics of the narrow, box-canyon site appear in this view from a suspension bridge near the axis of Mud Mountain Dam. Wall at left has been finished by high scalers. Note scaler's scaffold supports in lower foreground.

CONSTRUCTION OF Mud Mountain Dam, a flood control project on the White River in Washington, is going ahead under a revised design which substitutes a dumped, rockfill shell for the rolled earth fill originally contemplated; the core, as before, is to be rolled earth fill. The height remains at 425 ft.; a record for a structure of this type. The original contract with the Guy F. Atkinson Co. has been modified by deletion of embankment items, remains in force in other respects and is supplemented by an agreement on a cost-plus-a-fee basis covering construction of rockfill embankments. Unit prices in the competitive bids were published in *Engineering News-Record*, Sept. 14, 1939, p. 364.

The site of the project, which was authorized under the flood control act of June 22, 1936, is 47 miles southeast of Seattle, Wash., in a narrow, box canyon with vertical rock cliffs about 230 ft. high. The canyon is only 90 ft. wide at streambed level and 150 ft. wide at the top of the rock walls. Even with the widening slopes above the box canyon level, the structure has a most unusual combination of dimensions; crest only 700 ft. long with a base width of 1,600 ft.; with the flatter slopes that would have been used on an earthfill the base width would have been 2,200 ft. Function of the dam is to control destructive floods that have caused much damage in White and Puyallup valleys.

In addition to the dam itself (total fill under revised design 2,109,000 cu.yd.) the project includes a concrete-lined spillway and two concrete-lined outlet tunnels. The completed project involves about 4,000,000 cu.yd. of excavation and 60,000 cu.yd. of concrete.

The material that was to be used for constructing the dam as an earthfill structure lies in a deposit on the right bank 1½ miles from the dam. From this pit was to be taken a mixture of sand and gravel for the shell and the same material mixed with 20 percent of till for the core, all to be placed in 6-in. layers and compacted at near optimum moisture content. When the borrowpit was stripped it was found that the residual moisture content exceeded the optimum and also that mechanical grading showed an excess of fine material. Observations on a test fill 5 ft. high made at the borrowpit indicated ingredients and moisture content such that the test fill would neither drain nor dry so as to consolidate properly.

It is believed that the presence of a small percentage of colloidal clay in the borrowpit material is responsible for this failure of the test fill to drain adequately. Independent of test fill results, experience at the dam indicates that weather will be an important factor in construction—during a large part of the year the fill material would be "too plastic" to be placed and dry seasons are so short that many years would be necessary to complete the 3,000,000 cu.yd. of rolled fill originally proposed.

Decision that the material was not suited to the design under contract was reached June 15, 1940, at a meeting of officials and consulting engineers. Consideration was then given to washing out some of the fines and also to the possibility of drying out the materials to be used in construction.

Final decision by the U. S. Engineer Department on July 1, 1940, was that only the central core should be a rolled earthfill as originally specified and that the remainder of the dam shoud be built as a dumped rockfill, using rock from a quarry about 3 miles north of the damsite. A road has been constructed to this quarry, stripping is under way and equipment is being installed to excavate, deliver and place in the dam the 1,539,000 cu.yd. of rock which will support the central, rolled earth core with slopes and widths shown in the drawing. Total yardage in the dam will be 2,109,000 cu.yd.

Under the revised design of the dam, the cross-section is divided into the three major zones shown in the drawing. The one in the center (zone

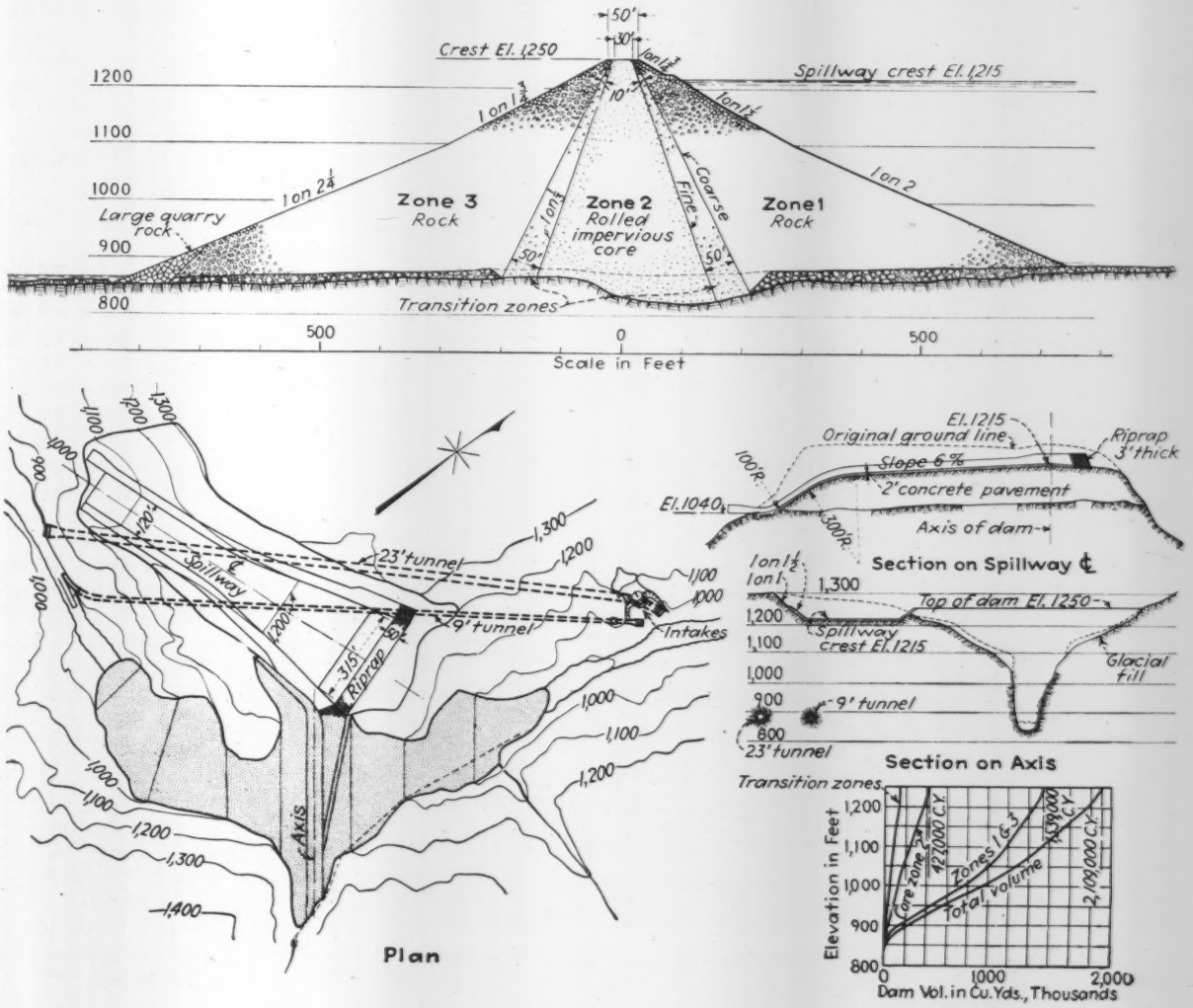
2) is a rolled impervious core composed of a mixture of sand and gravel from the original borrowpit, combined with 20 to 40 percent of the overlying blue glacial till. This zone is to be carried up with slide slopes of 1:$\frac{1}{3}$, flanked on either side by a transition zone in which the material will be graded from fine, close to zone 2, to coarser materials on the outer edges thus providing gradual gradation from zone 2 to the quarry rock of zones 1 and 3. In these two outer zones, the quarry rock will be dumped into place and sluiced down with $1\frac{1}{2}$ cu.yd. of water per cubic yard of rock. Zone 2 and the transition zones that flank it rest on bedrock in the canyon bottom. Zones 1 and 3, on the contrary, are founded on the natural streambed material.

Material for zone 2 is to be excavated and scalped of its large boulders at the borrowpit, hauled by rail to the damsite and there dried in a plant (located in spillway) consisting of three large dryers somewhat resembling the rotary heaters used in asphalt plants. This equipment will be assembled with screens and belt conveyors in an arrangement such that practically any combination of sizes and dryness of materials may be obtained.

From the drying plant materials will move in covered conveyor to bunkers at the canyon rim whence they will be lowered in 8-cu.yd. dump buckets to the bottom of the canyon by stiffleg derricks. There it will be picked up by carrying scrapers which will spread it on the fill ready for compaction in the method originally specified. The moisture content is expected to vary but it is believed that it can be kept at about optimum value. An extensive tent protection system has been designed and the plan is to keep the working area on the fill under this protective covering throughout the rainy season.

Two tunnels around the site have been constructed in the right abutment, one 23 ft. and the other 9 ft. in diameter. During construction the river is being bypassed through the larger tunnel. Even though it is lined with concrete, it was thought unwise to subject its lower portion to hydraulic pressure. Hence, when this tunnel is no longer needed for diversion purposes, a concrete plug will be put in about mid-length and from this point downstream the flow will be carried in three 8-ft. steel pipes. Inflow into these pipes is to be controlled by three 96-in. Howell-Bunger,

Cross-section of Mud Mountain Dam showing the design revised to utilize quarry rock, dumped in place in outer zones flanking a rolled-earth core. Plan and sections of the dam and its spillway with mass diagram. Note location of 9-ft. and 23-ft. tunnels.



free-discharge valves, the largest of their type ever built. They are to operate under a head of 340 ft. and are expected to give effective control of the flow at a much lower cost than would valves of any other type.

The 9-ft. tunnel originally was planned to be ungated as its function is to handle the normal streamflow. Later it was decided to provide a gate in order that a pool with a depth of as much as 100 ft. could be created at the dam when needed for clearing debris from the reservoir. A radial gate was decided upon and as this requires a square section, tunnel lining of standard circular steel pipe could not be used. The risk of erosion from heavy gravel and boulders carried through the tunnel as bed load made it necessary to provide a lining highly resistant to erosive action. This is necessary because once the tunnel is put in service it will not be accessible for repair. Accordingly, 87 ft. of the tunnel is lined with rectangular cast iron sections, each 4 ft. long, weighing about 14,000 lb. These have peripheral ribs on the outside to make effective bond with the concrete in which they are imbedded.

PRINCIPAL DATA ON MUD MOUNTAIN DAM

Height of dam above bedrock	425 ft.
Length at crest (excluding spillway)	700 ft.
Width at crest	50 ft.
Width at base	1,600 ft.
Volume of compacted fill	427,000 cu. yd.
Total volume of material in dam	2,109,000 cu. yd.
Drainage area	402 sq. mi.
Elevation of dam crest	1,250 ft.
Elevation of spillway lip	1,215 ft.
Lowest elevation of bedrock	825 ft.
Length of reservoir	5½ mi.
Reservoir storage capacity	130,000 acre-ft.
Maximum flood of record (1933)	28,000 sec.-ft.
Spillway capacity	139,000 sec.-ft.
Total capacity, 2 outlet tunnels	16,000 sec.-ft.

The control gate in the 9-ft. tunnel, located 63 ft. from the upstream portal structure, will normally be under a head of 100 ft. or less and will be accessible through a 100-ft. shaft.

The spillway is a concrete-lined channel over the right abutment with a capacity of 139,000 cfs. or about five times the largest streamflow of record. The spillway floor is surfaced with concrete 2 ft. thick poured in 30 x 30-ft. slabs. Joints between slabs are ¾ in. wide, made tight by copper waterstops. Edges of adjacent slabs rest on a 2 x 4-ft. concrete sill bedded in the layer of coarse gravel rolled into place as subgrade. Similar heavy paving is to be put on the spillway side slopes.

Contract plans and specifications for the project were prepared in the office of the division engineer, U. S. Engineer Department, North Pacific division, Portland, Ore., under the supervision of C. I. Grimm, head engineer; Col. Richard Park, division engineer; construction is under the direction of the Seattle district office, Col. B. C. Dunn, district engineer; Maj. Peter P. Goerz, construction engineer and Capt. James D. Lang, resident engineer.

The original contract awarded to the Guy F. Atkinson Co. in September, 1939, was for $5,344,605 and called for completion Nov. 15, 1942. Completion of the contract for the project according to the revised design is expected to cost about $8,000,000. The contractor's personnel at the dam includes: Ray H. Northcutt, resident manager, D. E. Root, chief engineer and general superintendent and Geo. W. Wintz, assistant manager.

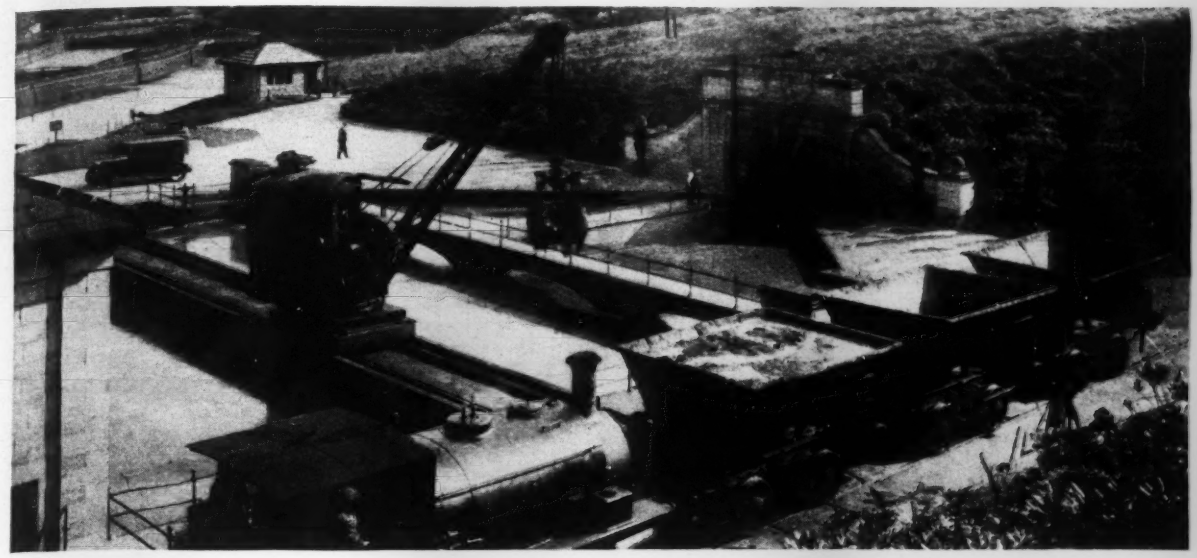
The drying plant and equipment are being built by the Madsen Iron Works of Los Angeles.

Streambed excavation underway following diversion of the White River through the 23-ft. diameter tunnel with upstream portal in the foreground.

Rochester Refuse Disposal

After one year's operation of the landfill method of refuse disposal, Rochester, N. Y., finds this practice highly successful, according to William H. Roberts, commissioner of public works. There have been no fires on the fill nor any complaints about the practice, despite the fact that operations are conducted on a site adjacent to a residential area.

Concerning the operation of the fill, Mr. Roberts reported to *Engineering News-Record* as follows: "Our practice has been to excavate a trench about 25 ft. in width and 6 to 8 ft. in depth in advance of the disposal of refuse. This trench is filled with refuse and then a 2-ft. cover of dirt is placed over the top of the trench. After the entire area has been filled in this manner, it is planned to use the original excavated material to again cover a layer of refuse, which will not exceed 6 ft. in depth. The trenches are excavated by the use of a 1-yd. dragline bucket attached to a 1¼-yd. gas-operated shovel."



Cleaning detritus tanks at the Bradford (England) sewage disposal works.

Grease from Sewage Helps Great Britain

H. Wontner-Smith
Sewage Works Engineer, Bradford, England

***Contents in Brief**—Wartime needs have created a big demand for grease in Great Britain. The Bradford sewage disposal works is supplying 500 tons weekly. This valuable byproduct, along with fertilizer, has been produced for almost four decades at Bradford, but this article reveals for the first time the commercial phase of this activity.*

ONE HEARS a good deal nowadays about recovery of metals, bones, rags, paper and so forth from household refuse, but here is the romance of the recovery of gold from polluted water —the golden stream from the washbowls of Bradford.

It has not been the practice of the Sewage Committee of the Bradford Corporation in the past to place before the public details of their business dealings, but now that markets are thrown wide open in these days of concentrated war effort, it may be an inspiration to many to learn something of the reaction of a municipal department when faced with what was at one time described in Parliament as an insoluble problem.

Many years ago, the growth of the wool washing industry in this famous but crowded center nearly spelled disaster to its inhabitants, because of the foulness caused by the decomposition of the fats and other organic matter discharged from the washbowls into the water courses within the city. Today one-fifth of the wool of the whole world and four-fifths of the wool grown and imported into England is washed and dealt with by this municipality. The wool-grease is present in globules in the water, some in emulsified form, with large quantities of soap and alkali used in the scouring of the wool.

It took a great number of years to discover ways and means of solving the problems connected with the recovery and marketing of this grease and afterwards, when success had been attained, it took many more years to complete the present Esholt sewage treatment works, which cost on completion more than $13,800,000.

At the Esholt plant, following grit removal, the sewage passes into detritus tanks (which retain finer mineral matter) and then into stormwater tanks. Here excess stormflow can be diverted while the balance (up to 64 mg.) passes to three 48-in.-dia. inverted siphons leading to the treatment facilities located some 2,100 ft. across the valley.

The siphons (two of which are in constant use, the third being kept in reserve) discharge into precipitation tanks where sulphuric acid is added to the sewage. The object here is to crack the soap in the sewage and precipitate dissolved organic matter, the wool waxes and the suspended matter being carried down in the process. A varying quantity of acid is required, ruled by the alkalinity of the incoming sewage; regulation is effected so that the amount of acid will produce a sewage which is about 10 parts per 100,000 parts acid, using methyl orange as an indicator.

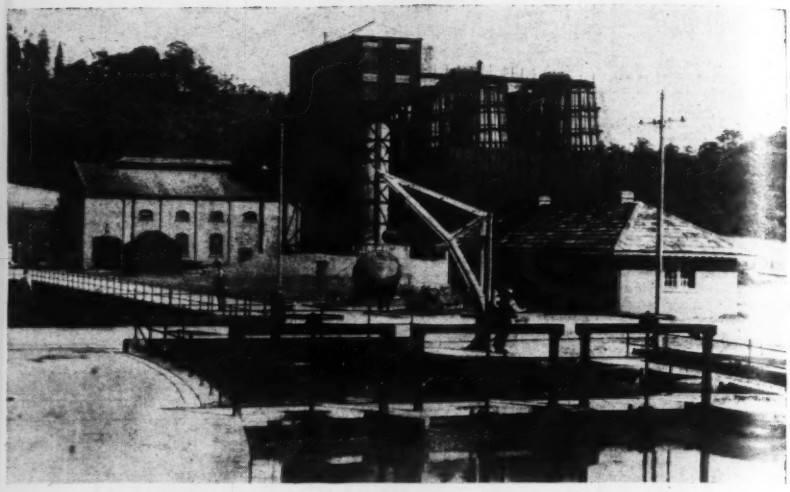
The effluent from the precipitation tanks is then settled in secondary tanks after which it is treated on bacterial filters. The filter beds are 6 ft.

deep and filled with coal, the latter ranging in size from ¾ to about 1¼ in.

The daily supply of sludge, which has a moisture content of 80–83 percent, is mixed with gritty sludge from the detritus tank in the proportion of 1 to 6; then it is passed to 7-ton vats, where it is heated by means of steam coils. The temperature is there raised to about 170 deg. F. at which point it tends to froth. When at the required heat the sludge passes to large displacement vessels from which it is forced by compressed air through 128 presses. Here the sludge is filter-pressed through cotton cloths by alternately feeding in hot sludge and superheated steam (to keep up the temperature). Liquid grease and water are discharged in the process.

The presses are operated for varying times according to the nature of the sludge to be pressed, but 64 hours' pressing is the usual practice. In that time the grease content of the cake is reduced to about 15 percent of the dry solids as compared with 40 percent in the original sludge, the water content of the cake being about 26 percent.

After the grease is treated for the removal of impurities it is barreled. Liquors resulting from the purifying of the grease and also the liquors passing from the presses, after separation of the grease, are returned to the precipitation tanks. The pressed cake is dumped on a drying area in the open where it remains for about 12 months to mature. Then the material is passed through a disintegrator which grinds the dried cake into a fine powder and the product finds a ready sale on the market for fertilizing purposes.



Sulphuric acid manufactured in this Mills-Packard chamber process plant is added to the sewage to aid in the recovery of grease at Bradford.

Large income from grease

There are no works anywhere in the world built on similar lines and no purification plants in existence which can claim an income in any way comparable with those of the Esholt plant. In the year 1903 it is recorded that 166 tons of grease was recovered, and the first sales realized $1,220. In five years the tonnage has grown to 1,606, with sales amounting to $57,800. In 1918 the tonnage became 4,524 with an income from sales of $633,490. In 1925 the yearly tonnage had risen to 6,271, and the income was $481,370. Up to this time, the old and now abandoned Frizinghall works were in operation and it was possible to recover only some of the grease. The remainder overflowed to the river.

The following year the new Esholt works came into play, but the grease was not saleable because of full markets. Moreover, with improved methods of extraction, the annual production of grease increased rapidly until, in spite of annual sales of about 6,500 tons, the stock had increased to tens of thousands of tons.

If these high quantities of grease had been put on the market at uneconomic prices, the losses to the corporation would have been immense. In order to widen the market possibilities a research chemist was appointed to survey the whole field covering the subject of wool fat and all its ramifications.

Press room where the sludge is treated for the extraction of grease and water, leaving a residue of value as fertilizer. There are 128 filter presses in operation.

Does research pay?

Sometimes, uninstructed people say that research does not pay. Here is a case where research has paid a thousandfold. One of the earliest results has been the manufacture of better forms of recovered grease. These clarified greases have found a place in the market, which to a large extent will remain for all time, and already more than $165,000 worth has been sold for purposes for which the ordinary quality of recovered grease would not in any case have been acceptable. In addition to this, nine other new products are being placed on the market, for several of which the demand is steadily increasing.

Thus, before the thought of war, steps had been taken. Now, with the advent of this war of unknown dura-

tion, the corporation finds itself in possession of probably the largest stocks of native grease in England.

The demand for this grease is such that, while sending out an average of 500 tons per week, the works are more than one month behind deliveries. The biggest demand is for the new and clarified greases. Of course, one cannot say how long this accelerated business will go on, but it would appear that while the war lasts and for a considerable period afterwards this grease will be in full demand.

In the last financial year (ending March 31, 1940, grease sales exceeded $400,000 although the full effect on world trade has not yet been felt. As to this year, an amount several times this figure is quite within the bounds of possibility.

Fertilizer production

Another product from the treatment process is organic manure which, although perhaps not of so spectacular a nature as grease, yet none the less is of great value in these times. This material is a fibrous though friable powder of low moisture content (about 15 percent), the humus portion being about 50 percent of the whole.

The manure is entirely sterile, containing no seed life or germs of any sort, and includes the valuable organic nitrogen content of the fine wool fibre. Some 30,000 tons of this material is available every year and is used largely for the dressing of grasslands, for which purpose it gives very excellent results. Golf clubs use it largely for the dressing of both greens and fairways and the price delivered to any club in England is $7.50 per ton. Also, public schools, sports grounds and farmers use considerable quantities, and compounders of manures take steady amounts. The peak year in sales of this material was in 1921 when more than $68,000 was realized.

The sewage committee has been asked by several transport companies why it is that Bradford does not supply among its products methane gas, as do many sewage disposal plants. The digestion of sludge produces a gas rich in methane and Bradford could, if they wished, have laid out their works to produce it. Such a procedure, however, would have resulted in heavy losses to the corporation, as both the grease and manure values would then have been affected without any compensating advantages.

Another matter of public interest, and of special moment now, is the question of sulphuric acid supplies. It may not be generally known that the sewage committee at Bradford is probably the largest user of this commodity in England in one individual works. An annual usage of 20,000 tons is recorded and sulphuric acid is a vital necessity. In order to stabilize the price some years ago, the committee erected a plant providing 9,000 tons annually. Now, in response to urgent government appeals, the committee has decided to double this plant. This will have the effect of securing for the department adequate supplies, while releasing for government use the additional supplies which were obtained from other sources.

While this is a review of some of the main business dealings of the department, it would not be complete without mentioning the vast volumes of very strong textile and other waste waters, the purification of which is the principal object of the committee's existence. The annual volume amounts to as much as 8,400,000,000 gallons, a quantity which no one, probably, can fully appreciate. Nevertheless it is not the volume but the quality and strength which has caused works of such complexity to be employed for purposes of wastes purification.

And it can be said with confidence, now that all surrounding townships, with similar trades, have agreed to join these central works, this part of the world, at any rate, will be a better place in which to live.

Hints on Distribution System Maintenance

READINESS for emergencies and consideration of the convenience of consumers are two requirements in handling distribution system repairs, according to F. H. King, assistant manager of the Peoria (Ill.) Water Co., who addressed a recent meeting of the Illinois Northwestern Water Plant Operators Conference.

Maintenance of distribution systems is governed to a considerable extent by the daily care given and the character of construction, said Mr. King. There should be a trained maintenance crew, with a foreman specially qualified to analyze troubles and start repairs promptly. Pipe and fittings must always be on hand, along with sleeves of all sizes and several sizes of repair clamps. Also there should be a pipe cutter, a ditch pump (preferably gasoline-driven) and a paving breaker; floodlights are desirable, especially at the larger plants. A pipe locator and leak detector will soon pay for themselves, and a truck must be available for use day and night.

Repairs should be made with as little inconvenience as possible to consumers. To avoid criticism when water is turned off for such work Mr. King's company uses sterilized milk cans for delivering water to consumers whose supply is affected. The men delivering these cans wear uniforms and make numerous calls to insure that the users have adequate water for their needs. Laying temporary pipe lines on the ground to supply large consumers is a service that has been well received. For this purpose a line is connected to a fire hydrant located beyond a closed valve and a 2-in. line is run to the consumer's premises; here a connection is made to a hose outlet or some other point, and the curb box is then turned off to prevent the water from flowing out into the main.

Valves and valve boxes should be inspected at least once a year, including complete opening and closing of the valve. Hydrants must be inspected to see that drips are free and giving proper drainage. A valve on a hydrant branch is desirable, especially in high-value districts, as repairs can then be made to the hydrant without shutting off any part of the water distribution system.

Sterilizing mains after repairs is important, as dirty water will find its way into the main and ordinary flushing will not eliminate the danger of pollution; chlorinated lime can be used for such sterilization. Mr. King emphasized the special importance of keeping complete and up-to-date records of mains, hydrants and valves, including adequate description and notes of all inspections and repairs.

Riverfront Boulevard for Charleston

C. P. Fortney
Consulting Engineer, Charleston, W. Va.

***Contents in Brief**—Five-mile highway on bank of Kanawha River in West Virginia city is arterial of most modern type. Reduces traffic congestion in a city confined between hills and a river. Six-lane street through business section becomes two three-lane divided roadways in residential districts. Contains a bridge with a 250-ft. girder span. Municipal parking lot a part of project.*

KANAWHA BOULEVARD, recently completed, affords the residents of Charleston, W. Va., their first modern thoroughfare which, extending five miles along the Kanawha River from one end of the city to the other, greatly relieves the congested traffic problem caused by rapid growth of the city in the limited space available between the river and a nearby range of hills. The first mile of a 50-ft.-wide boulevard, including a bridge over the Elk River, was started in 1937 with $1,000,000 provided by a PWA grant and a city bond issue. Before this project was completed, $2,500,000 additional was raised to finance another four miles, the width of which was increased from 50 to 60 ft. The entire project is now in service and plans are underway by the state highway department to extend the boulevard 1.1 miles east as part of a grade-crossing elimination project, thus strengthening the view that some day the boulevard may run for miles through the industrial sections of the great Kanawha River Valley.

Preparing the right-of-way

Building Kanawha Boulevard involved three rather distinct operations —preparing the right-of-way, constructing the roadway and erecting the Elk River Bridge—each of interest in itself. Dredging, grading and riprapping constituted the right-of-way preparation, an operation whose size may be gaged by the facts that dredged material for the necessary fill totaled 1,300,000 cu. yd., and that the contractor's plant included ten derrick boats, a fleet of barges and two tow boats. Not only did the dredging furnish the required fill but it improved river navigation by removing a great bar, which had formed where the Elk River empties into the Kanawha.

The type of embankment used is shown in Fig. 3. Construction began with the toe wall, excavated 8 to 20 ft. to solid foundation and built up with quarried stones to an elevation 2 ft. above the normal pool. In all, 130,000 cu. yd. of stone was required. The area behind this wall was filled with selected material, placed in layers on the original riverbank, which had been terraced by a bulldozer.

After the embankment had been brought to sufficient elevation, it was graded for riprap and covered with a layer of gravel ranging in size from coarse at the surface to fine at the bottom, a type of construction designed to prevent the embankment from washing beneath the riprap stone. Large derrick stones were then laid on this gravel bed and "chinked"

Fig. 1. Kanawha River waterfront in Charleston, W. Va., before and after the new boulevard was built. The state capitol dome rises in the distance.

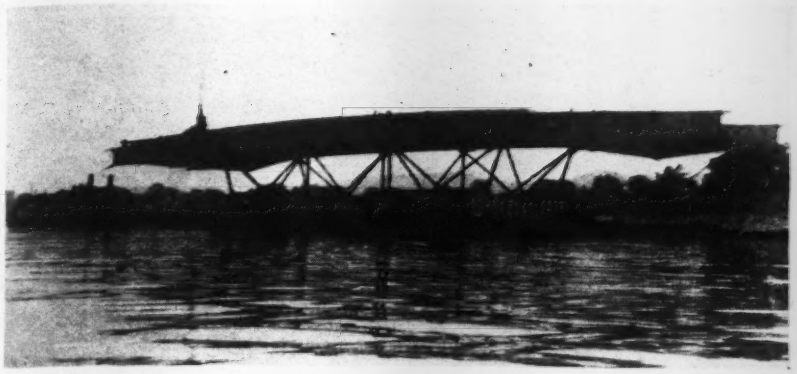


Fig. 2. Floating the Elk River Bridge to the site. The assembly includes the 250-ft. center span and 50-ft. sections of the 125-ft. end spans.

by closely driven quarry spalls, giving a uniformly paved surface.

The riprap, of which there is 57,500 cu. yd., was placed on the slope to 25.5 ft. above normal pool, which is above the average high water stage. At this elevation the bank was recessed 10 ft. to accommodate a 5-ft. walk, flanked on one side by a white curb and gutter and on the other by a 3-ft. strip of sod. The walk, sod, curb and gutter tend to drain the surface water from the upper part of the embankment and thus prevent scouring behind the riprap. Besides serving this purpose the terrace affords a pleasant place to walk along the river, and it improves the general appearance of the project. The remaining embankment is protected by sodding, of which there is 100,000 sq. yd. in all. Stone and concrete steps lead from the street to the shoreline at specified intervals, an elaborate boat landing being a feature in front of the state capitol building.

Constructing the boulevard

The boulevard design provides for two 27-ft. roadways for one-way traffic and a center dividing strip, generally 6 ft. wide and raised and protected by curbs. The 27-ft. roadways consist of two 10-ft. driving lanes and a 7-ft. parking lane. This design is altered throughout the business district, where six 10-ft. lanes without a center dividing lane are provided. White concrete lane markers, poured with the slabs, were used to eliminate the maintenance expense of painted lines.

Concrete safety curb

It was necessary along the entire length of the north side of the boulevard to protect the motor traffic from driving over the embankment into the river. This could have been accomplished by a wall or guard rail but neither would harmonize with the simple lines of the boulevard. Therefore, an 18-in. reinforced concrete safety curb was used, and has proved very satisfactory. As shown in one of the illustrations, this north edge construction is varied at one point in the business section where parking facilities are provided on the embankment.

To meet the time requirements of the PWA and to accommodate traffic it was often necessary to pave immediately after filling. Foreseeing the possibility of some settlement, mudjack sleeves were placed in the slabs as they were poured. Settlement, however, has been very slight, and there has been little need for the mudjacks.

Although the boulevard runs through a business and residential district, sodium vapor lighting is used, because of the sometimes foggy atmospheric conditions along the river. The luminaires are rated at 10,000 lumens and are staggered at 100-ft. intervals. The height from the center of the light to the pavement is 25 ft.

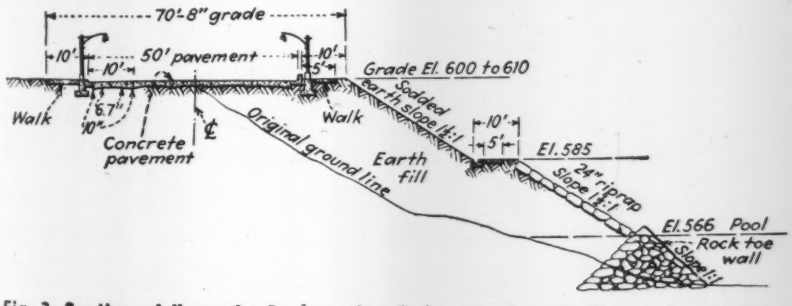


Fig. 3. Section of Kanawha Boulevard and river bank protection. Later construction has utilized a 60-ft. pavement and a dividing island outside the business district.

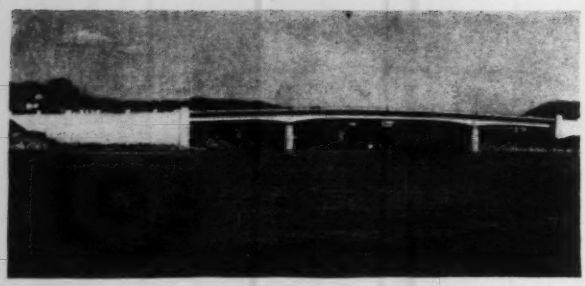
A 250-ft.-span girder bridge

The bridge over the Elk River has two piers built on a rock foundation 21 ft. below the normal pool stage. Counterforted retaining walls, 20 to 40 ft. high, form the east and west abutments; foundations are steel H-piles.

The superstructure is a three-span continuous plate girder bridge with side spans of 125 ft. and a center span of 250 ft. War Department requirements for an opening of 240 by 35 ft. clear determined the center span dimension. Two main longitudinal supporting girders 9 ft. 10½ in. deep at the ends and center and 14 ft. deep over the piers were used. These were designed for an H-20 highway loading on five lanes and for two 8-ft. cantilever sidewalks. The roadway is a 4¼-in. concrete-filled steel grid with 1½-in. asphaltic concrete wearing surface. Dead load, increased by 20 percent of itself, was used for calculating the camber.

The steel erecting contractor elected to assemble the center span and float it into position. Dock facilities were leased about two miles below the bridge site, where the center span and 50 ft. of each side span was assembled on two steel barges; the remaining 75 ft. of each side span was utilized in the falsework. With steel erection completed, the bridge was towed to the site and lowered onto the piers by pumping water into the barges. An A-frame erected upon the bridge was used to lift the remaining girders into place. The dead load reactions of the steel girders at the main river piers were weighed by hydraulic jacks and adjusted as necessary by shims.

The original purpose of the boulevard to relieve traffic congestion has



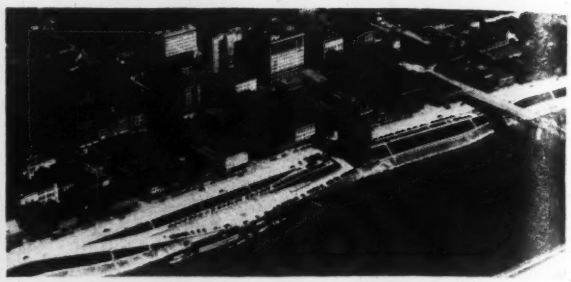


Fig. 3. Two features of Charleston's riverfront boulevard are the Elk River Bridge (left), which includes a 250-ft. plate girder span, and the parking facilities (right), which are provided in the business district.

been accomplished. For the motorist routed through Charleston by any of the three federal or several state routes, the time required to go from city limits to city limits has been reduced from thirty to ten minutes. Besides traffic relief, the appearance of the waterfront has been improved immensely. The riprap which protects the bank from rapid shifts of the Kanawha River and prevents erosion has been tested by several spring floods, proving satisfactory.

Grading contract on the work was held by the Dravo Construction Co., Pittsburgh, Pa. Stellhorn and Beichtler, Columbus, Ohio, had the paving contract, and the Mount Vernon Bridge Co., Mount Vernon, Ohio, the steel contract on the Elk River Bridge. The writer was consulting engineer.

Strengthening the Frame of a 27-Year-Old Concrete Building

C. P. Lewellen
Designing Engineer, Howard R. Green Co.
Cedar Rapids, Iowa

***Contents in Brief**—In the public eye 27 years ago because of collapse, the Stark-Lyman Building, Cedar Rapids, Iowa, now claims attention through a strengthening procedure to increase its load-bearing capacity. In the remodeling process columns were incased in concrete, spans were cut down by steel beams and additional steel columns. Shrinkage problems and the transfer of column bearing loads required special construction methods. Hoops with hooks, instead of a spiral, reinforce the new shells around the columns.*

Twenty-Seven Years Ago the engineering profession was shocked by the collapse during construction of part of the reinforced-concrete Stark-Lyman Building in Cedar Rapids, Iowa. Two rear bays of the structure fell while concrete for the roof slab above the seventh floor was being placed. It was a typical cold weather failure resulting from premature removal of forms following freezing. (*Engineering News*, Nov. 27 and Dec. 18, 1913 and Jan. 8, 1914, and in *Engineering Record*, Nov. 22, Nov. 29 and Dec. 27, 1913.) Recently when it was sought to change the original light occupancy of the building to permit the Century Engineering Corp., Manufacturers of household heating boilers and oil-burning equipment, to occupy the whole structure, reinforcement for the heavier loads was undertaken. The manufacturing company, long a tenant, wished to buy the building in case it could be sufficiently strengthened. The reports in the two predecessors of *Engineering News-Record* were about all the engineering design data which were found to be available.

The building is 140 ft. long by 60 ft. wide and seven stories high. Columns are spaced at 20-ft. intervals each way, making three bays in the width of the building and seven bays in the length. All columns are of reinforced concrete, spiral reinforcing with longitudinal steel having been used generally to the fourth floor while longitudinal reinforcing with lateral ties was used above this floor. Girders built monolithically with the floor system run crosswise of the building on the column center lines. Concrete joists alternating with clay tile span between the girders forming a floor reinforced in one direction and acting with continuity across the tops of the supporting girders. Spandrel beams carry the brickwork at the exterior faces of the structure.

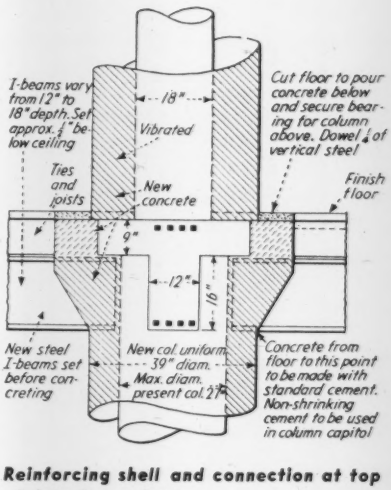
New load requirements

Load requirements for use of the building as planned by the buyer called for a nominal load on the two north bays of the first floor to provide display room quarters and 100 lb. per sq. ft. on the balance of the floor for shipping room purposes. The entire second floor was to be used for offices with the exception of one panel

where records were to be stored. The minimum design live load permitted under the city building code is 50 lb. for office occupancy and 100 lb. for the record storage room. Machine shop loads of 100 lb. in the center bays of the third and fourth floors were required with 70 lb. in each adjacent half bay for the temporary storage of small units under manufacture. Passageways located at the exterior walls reduced the load requirements in the outer halves of the exterior bays. Assembling of equipment on the fifth floor called for 70 lb. over the entire area except for the outer halves of the exterior bays where passageways were to be maintained as on the third and fourth floors. Storage space was to be provided on the sixth and seventh floors with a load requirement of 70 lb. in the center bay and 25 lb. necessary in exterior bays.

The former owners of the building believed that it was sufficiently strong to carry these loads, and to satisfy the prospective purchaser they desired to have load tests made with the expectation of substantiating this belief. Because of the history of this structure it was suggested that a review be made of the published reports and such plans as were on file in the office of the city building inspector to determine whether or not such loadings as were required could be safely carried, using methods of analysis usual in present reinforced concrete design. A physical examination of the building was also necessary to supplement information lacking upon the plans and to disclose any evidence of overstress from loadings incurred during previous occupation of the building.

Frame strengthening of this 27-year-old building permits manufacturing occupancy.

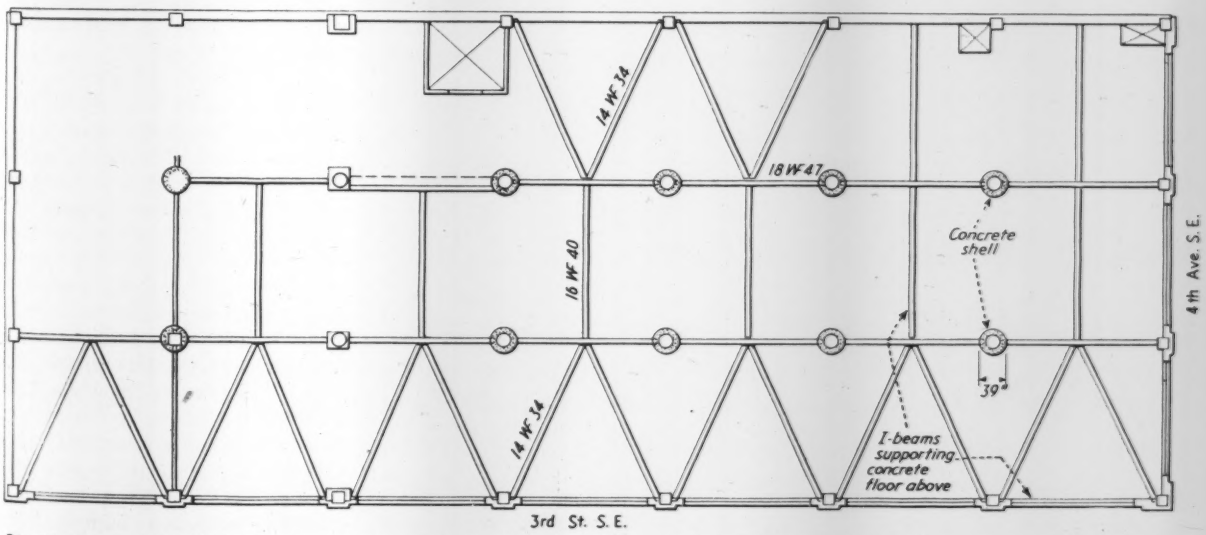

I-beams vary from 12" to 18" depth. Set approx. 4" below ceiling
Ties and joists
18"
Vibrated
New concrete
9"
12"
16"
Cut floor to pour concrete below and secure bearing for column above. Dowel of vertical steel
Finish floor
New steel I-beams set before concreting
New col. uniform 39" diam.
Max. diam. present col. 27"
Concrete from floor to this point to be made with standard cement. Non-shrinking cement to be used in column capitol

Reinforcing shell and connection at top of interior columns.

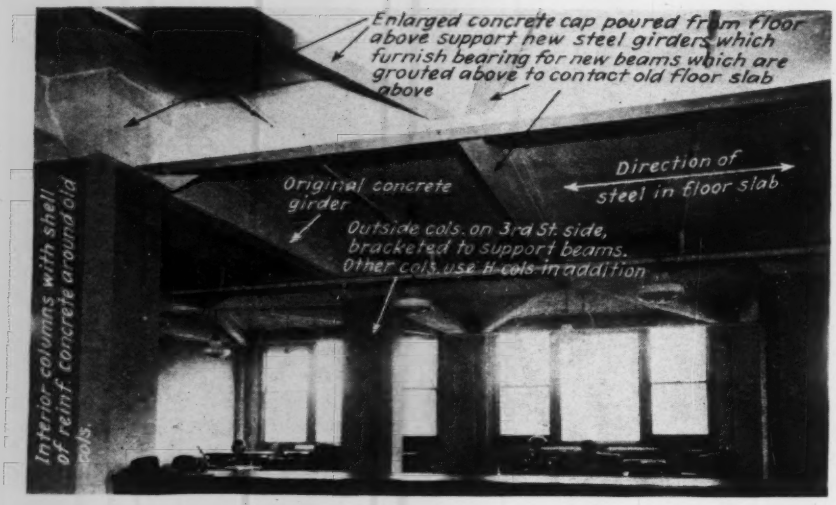
Condition of the building

This examination showed a finished floor 1½ in. thick had been placed upon the structural slab on all floors. Ceilings and columns had been plastered except in the basement and on the seventh floor. In many instances shear cracks were visible in the plaster on the sides of the floor girders near the interior columns from the first to the seventh floors. No indication of similar weakness was found in the beams supporting the first floor. Also there was an absence of cracks in the ceilings which indicated joist deflections were not excessive from the storage loads then upon the floors as the plaster had been placed directly against the underside of the tile and concrete joists. A rough computation of the live load being carried at this time showed it to be 50 lb. although one panel in the second floor located in the reconstructed section was loaded with fractional horsepower motors in cartons piled seven high which figured approximately 175 lb. per sq. ft.

The concrete in the reconstructed section seemed to be tough and produced a good clear ring when struck with a hammer, which was not the case with the concrete in the balance of the building. Generally the concrete spalled readily under a chisel struck with a hammer and in some instances large pieces of concrete



Steel beams were added to reduce reinforced concrete spans in Stark-Lyman building and columns were strengthened by incasement in concrete shells.


Enlarged concrete cap poured from floor above support new steel girders which furnish bearing for new beams which are grouted above to contact old floor slab above
Original concrete girder
Direction of steel in floor slab
Outside cols. on 3rd St. side, bracketed to support beams. Other cols. use H-cols. in addition
Interior columns with shell of reinf. concrete around old cols.

Features of the reconditioning work as it appears on the second floor ceiling.

could easily be broken up with a few hammer blows, showing a lack of desirable cohesion between particles. No cores were drilled or compressive strengths of concrete determined.

From data available the working stresses apparently used in the original design were as follows: Steel in tension, 20,000 lb. per sq. in.; concrete in bending compression, 750 lb. per sq. in.; concrete in direct compression (columns), 600 lb. per sq. in. The 750-lb. compressive working stress for concrete under flexure would indicate a 2,000-lb. concrete as it was customary to use three-eighths of the ultimate crushing strength of concrete for the allowable unit stress at the time of the original design. Upon this premise $n = 15$, $k = 0.36$, $j = 0.88$, and the allowable unit shear in the concrete would be 40 lb. per sq. in.

From calculations it was found that the design was not well balanced and fell considerably short of providing the 95 lb. per sq. ft. live load capacity claimed under the original specifications. The computed safe live load strength of the floor girders is summarized as follows: 70 lb. based on steel stress, 27 lb. based on concrete compressive stress, 18 lb. based on concrete shearing stress. These values compared with those of the tile joist system show the floor to be somewhat stronger than the supporting girders. This condition was indicated visually by the previously mentioned shear cracks near the support of some girders.

After consideration of the above analysis it was apparent that live loads of 70 lb. to 100 lb. per sq. ft. could not be safely carried without some additional strength being provided. Accordingly the idea of performing load tests was abandoned and a plan of using additional floorbeams as well as column strengthening was developed. From the review of the original plans no floor strengthening was required for the first floor as it is of much heavier construction than the upper floors. Reinforcement was indicated for the entire second floor, but in this particular case approval had been given by the city building inspector's department in past years for the use of this floor by loading at least equal to or perhaps greater than the 50-lb. proposed without evidence of the floor being unsafe. Because of this experience and the fact that practically the entire space would be open in one general office with an actual load of much less than 50 lb., no strengthening was deemed necessary except to provide capacity for 100 lb. for the slab of the record room. The third to fifth floors inclusive required strengthening in all panels while the sixth and seventh floors needed it only in the panels of the center bay to satisfy the loading conditions.

Analysis of stresses in columns under the proposed loadings was impossible due to the lack of specific information regarding reinforcement and it was decided to eliminate this uncertainty by incasing all interior columns with an additional shell of 6 in. minimum thickness of concrete containing spiral and vertical reinforcement, and to prevent additional load on the exterior columns by the use of auxiliary steel columns where strengthening of exterior bays was required. The nature of occupancy made the use of steel beams and columns unobjectionable except on the second or office floor where enclosure with metal lath and plaster was considered advisable.

Recommended remodeling

The plan of remodeling as recommended contemplated the use of steel beams placed directly beneath the ceiling at right angles to and at the midpoint of the joist spans to cut their length in half. This plan would provide more than the required strength in bending so far as the joists were concerned although cracks were expected to open above the steel beams due to lack of negative reinforcement at this point. This was not considered serious as it in no way affected the safety of the floor. Under this system the unit shearing stress in the joists was reduced to 32 lb. under a live load of 100 lb. The ends of the transverse steel beams in the interior bay and the interior ends of steel beams in the outside bays were to be supported by longitudinal steel beams framing into the columns and supported upon the incasing shell of concrete for column strengthening. The exterior ends of the transverse beams in the outside bays were to be supported by a steel header beam in turn resting on steel columns adjacent to the concrete wall columns along the east side and two panels of the west side of the building where window or door openings could not be obstructed; otherwise steel columns were to carry this load direct from floor to floor.

All of the recommendations were carried out in the final design except for the location of the beams supporting the joists in the exterior bays and elimination of the header beams and their supporting steel columns at the exterior walls. The exterior header beams proved to be undesirable from the standpoint of appearance as they would have extended below the tops of the windows and would be particularly objectionable at the ceiling of the office floor. To overcome this difficulty two beams were used under the exterior panels of the third, fourth and fifth floors except where windows did not interfere. These beams were placed in a V-shape with their junction at the apex of the V supported at the center of the longitudinal beams

framing into the interior columns, and the outer ends supported on the concrete wall columns by means of a steel collar recessed into the concrete. Such an arrangement gives a variable load-carrying capacity to the floor that is usually undesirable but in this case was the only one acceptable to the owner.

Column Loading

Steel columns used to support steel floorbeams at the north and west sides of the structure were equipped with bearing plates at each end; the top plate was in direct contact with the bottom flange of the steel beam and a grout cushion beneath the lower plate transferred the load to the flange of the concrete spandrel beam which in turn was supported by a steel beam and column loads transferred from floor to floor were carried down to foundations resting upon solid rock. Vertical and spiral reinforcing placed in the shell of concrete around the existing columns was determined by designing theoretical columns of area sufficient to carry the entire load, assuming 3,000-lb. concrete, and using one percent of this area for vertical steel and one-half percent of the area for spiral steel.

Some concern was felt in the matter of transferring column loads through the floors from the new column shell above to that below the floor. It was to provide some continuity of vertical steel and concrete bearing at this point that large capitals were used, whereby tile could be removed between the joists on each side of a column and the concrete poured up to the top of the floor with dowels projecting for bonding in the next tier of columns. Shrinkage of this concrete was also given consideration as it was necessary to have it in intimate contact with the underside of the floor girders to prevent the stem and flanges from being subjected to undue shear from direct column load. The specifications called for the concrete to be placed from the floor to within a few inches of the ceiling and allowed to stand 24 hours permitting shrinkage to take place, then continuing up through the next floor using a so-called non-shrink type of cement.

In construction it was found that the bond between the new concrete and that of the old columns prevented this shrinkage from becoming cumulative. High-early-strength cement was used in order to release forms as soon as possible. Because of the difficulty of wrapping bars around the existing columns, the spiral reinforcement was not placed as a true spiral but as hoops with hooks on each end 16-bar diameters long. Care was taken to stagger the spacing of the hooked ends of the hoops. In general column concrete was placed from the floor to the bearing of the steel floorbeams which was also the bottom of the capital. After erection of the beams concrete was placed in the capital through openings in the floor above.

At the floorbeam locations the plaster was removed from the underside of the slab for the width of the beam flange. The beams were then set 1 in. below the bottom of the slab and positive bearing of the joists upon the beams insured by filling the space between top flange and underside of floor with cement mortar at a time when no load was imposed upon the slab immediately above.

Organization

The general contractor for the remodeling was the Paulson Construction Co. of Cedar Rapids. The work was completed for $18,380 within 90 days without any unusual construction difficulties. The total cost of the improvement including steam piping, installation of sprinkler system, office partitions and other work not covered by the general contract was $30,000. Engineering work in connection with the revision was performed by Howard R. Green Co., consulting engineers of Cedar Rapids, Iowa. The net result of the improvement was to more than double the value of the building which has an inclosed volume of 858,800 cu. ft. and an estimated present replacement value depreciated of $220,000.

Prestressed Concrete Water Tanks

***Contents in Brief**—Low-cost water storage in areas served by the East Bay Municipal Utility District is provided by eleven circular concrete tanks ranging in capacity up to 3½ mg., now under construction or built in recent years.*

CIRCULAR reinforced-concrete tanks with prestressed steel bars around the outside of the walls are an economical means of providing water storage and equalizing capacity in the hilly terrain served by the East Bay Municipal Utility District at Oakland, Calif. Steep hillsides make the excavated type of reservoir impracticable and because this is a high-class residential district, the concrete tanks, which lend themselves well to architectural treatment, are much preferred to any other kind of above-ground storage. Beginning with a 200,000-gal. tank in Upper Cragmont, this type of construction was successful from the start. As experience was gained and improved methods of design and construction were developed, larger and larger sizes were built. The recently completed Carisbrook tank is 160 ft. in diameter and has a capacity of 3½ m.g.

Significant features of the design

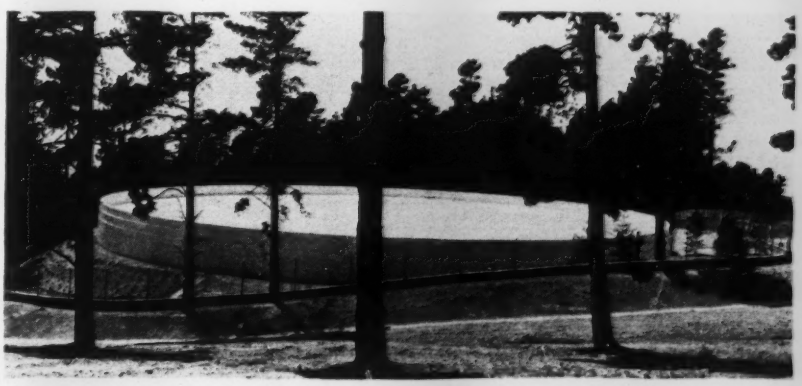


Fig. 1. Completed Carisbrook tank after backfilling and landscaping.

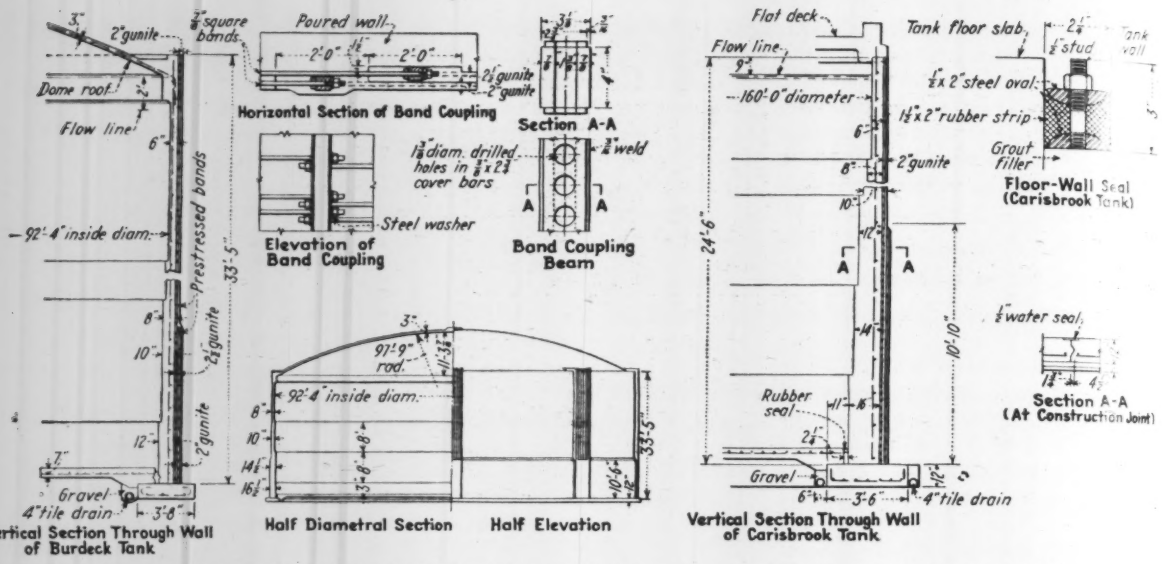

Vertical Section Through Wall of Burdeck Tank
Horizontal Section of Band Coupling
Section A-A
Elevation of Band Coupling
Band Coupling Beam
Half Diametral Section
Half Elevation
Vertical Section Through Wall of Carisbrook Tank
Floor-Wall Seal (Carisbrook Tank)
Section A-A (At Construction Joint)

Fig. 2. Typical details in circular tanks with flat roof (as at Carisbrook) and with domed roof (as in the Burdeck tank).

are: (1) a method of constructing the walls in separate vertical segments or panels poured alternately to simplify forming and to permit each vertical segment to be completed in a single pour, (2) a simple method of prestressing the circumferential bands in 40-ft. lengths terminating in welded steel box-columns where exactly the same stress can be put in each band and, finally, (3) means of making a bottletight joint between the floor slab and the side walls. These methods, performed repeatedly under the same conditions until the practice became standardized, have so simplified construction that tanks recently built have been put up exclusively by WPA labor with only one foreman and one inspector provided by the district.

Typical sections and details for both flat-top and dome-shaped roofs are shown in the accompanying drawing. The torque-limit wrench

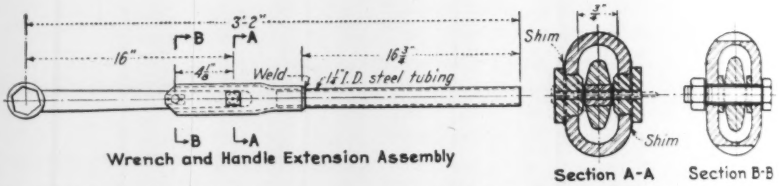


Fig. 3. Details of prestressing wrench used in tightening steel bands.

shown in Fig. 3 was designed expressly for the requirements in prestressing the bands. An accompanying table gives capacities and construction dates for the eleven tanks that have been completed or are now under construction by the East Bay District.

Ordinary construction methods are used in preparing the foundation and pouring a reinforced circular floor slab whose outer edge rests on the deeper footing placed for the circular wall. The enclosing wall in the larger tanks is built in ten to eighteen segments, each of which is formed separately and poured as a monolithic panel. Forms are set up first for alternate panels, the intervening panels being poured later. The vertical joints between panels are sealed on the inner face by a ½-in. depth of mortar made of cement and iron dust, troweled on after the prestressing of the tank walls has been completed.

Wall thickness varies with height and diameter of the tank. In the largest tank thus far built (Carisbrook) wall thickness ranges from 16 in. at the bottom to 6 in. at the top. Poured concrete portions of the wall have sufficient ½-in. reinforcing bars (mostly spaced 18 in. each way and im-

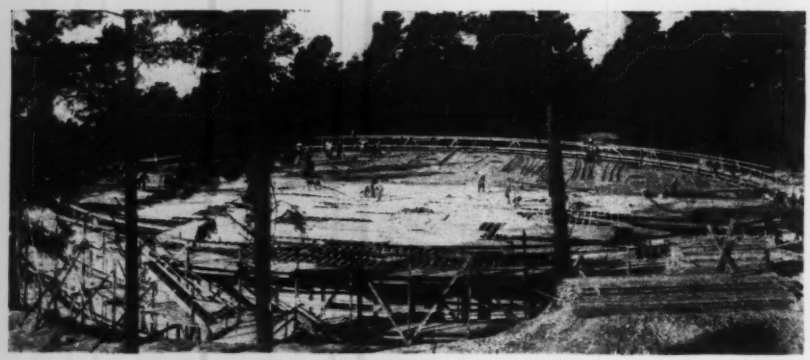


Fig. 4. Laying sheathing for the flat roof slab in the 3½-mg. Carisbrook tank.

PRESTRESSED CONCRETE TANKS BUILT BY THE DISTRICT

Name	Date Completed	Capacity (gal.)
Upper Cragmont*	1934	202,000
Orinda Wash Water*	1934	200,000
Crockett	1936	1,000,000
Berkeley View	1937	1,000,000
Lower Cragmont	1938	741,000
Montclair	1938	1,516,000
Carisbrook	1940	3,500,000
Burdeck	Under construction	1,500,000
Arlington	Under construction	3,000,000
El Portal	Under construction	3,000,000
Joaquin Miller	Under construction	3,000,000

*These two have walls built up with shotcrete; all others have poured walls.

bedded 2 in. from inner face) to strengthen the individual segments pending completion of the entire circle.

After the circular wall has been completed by the pouring of all segments, concrete is allowed to set for seven days and then the principal wall reinforcement consisting of $\frac{7}{8}$-in. square, circumferential bands is put on. These bands go on in two layers; the inner layer bears directly against the outer surface of the poured concrete wall and after the individual bands have been stressed and stresses readjusted a $2\frac{1}{2}$-in. thickness of shotcrete is put on. When this has set for seven days the second layer of bands around the lower portion of the tank goes on outside the shotcrete surface. Bands of the second layer, in turn, are stressed and then a final shotcrete coating is put on to give them a 2-in. cover. The bands are cut in lengths such that the joints can be made in band coupling beams spaced at 30-deg. intervals around the circumference. Vertical spacing between bands varies from 4 in. at the bottom to 17 in. or more at the top of the larger diameter tanks.

The box beams used for coupling the bands are made up by welding two $\frac{3}{8}$-in. facer plates to the edges of two $\frac{7}{8}$ x 4-in. bars. The beams are placed vertically at the coupling points with the flat side against the poured concrete surface as shown in Fig. 2. The circumferential bands are open-hearth, controlled, fine-grain steel with a minimum yield point of 55,000 lb. per sq.in., minimum tensile strength 85,000 lb. per sq.in. and a minimum elongation of 17 percent in 8 in. A 5-in. length at each end is upset to $1\frac{1}{2}$-in. diameter and threaded for $4\frac{1}{2}$ in. Threads are right-hand on one end and left-hand on the other. After threaded ends of the bars are passed through the coupling beams, steel hexagonal nuts are put on and the bar is prestressed by use of a torque-limit wrench in which a 6-d. copper nail (of known shearing strength) is sheared off in applying the final tension. This method of prestressing is far superior to the plan, initially tried, of using turnbuckles. The flat surface of the square bar also gives better bearing against the concrete than was possible with circular bands.

Beginning at the bottom and working upward on successive bands, the prestressing is adjusted as closely as possible to 25,000 lb. per sq.in. except that in tanks which have a dome roof the seven bands at the top are prestressed to 30,000 lb. per sq.in. after the dome concrete has cured for seven days. Stressing any one band usually affects stresses in neighboring bands, hence several adjustments of each band usually are made and frequently more than one copper nail is sheared to make sure of the tension. The threads are well greased before the nuts are put on and the pressure is first run up on all nuts in a set of bands with a pull of about 150 lb. applied at the end of a standard wrench. Final adjustment is always made with the special limit-torque wrench.

When the wall segment forms are set up care is taken to locate them so that the inner face of the tank will be $2\frac{1}{4}$ in. from the outer periphery of the floor slab. After the forms are stripped, to maintain this $2\frac{1}{4}$-in. annular space, wedges are driven in at 10-ft. intervals and are left in place until just prior to *final* adjustment of band stresses. After prestressing the bands, the wedges are taken out and the sealing recess is filled, to a depth of 9 in., with firmly tamped grout. On top of this grout a $1\frac{1}{2}$ x 2-in. strip of pure gum rubber is pressed down into the upper part of the annular space. By means of steel ovals above and below the strip with bolts running through it at 6-in. intervals around the periphery, the rubber can be compressed so as to force the sides of the strip against the two concrete surfaces. Where this is done carefully after the final stressing on the bands around the tank, the joint becomes practically bottletight. Similarly, the vertical joints between segments have been found to be entirely satisfactory and no unsightly seepage has appeared on the outer surfaces to mar the appearance.

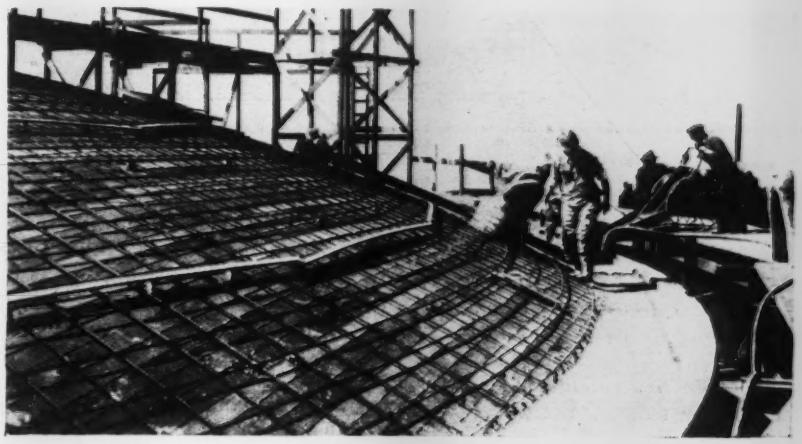


Fig. 4. Pouring concrete in a dome-shaped roof over a 1-mg. circular tank.

Dome roofs have been used by the district for all tanks up to 92-ft. dia. In a dome of this span the rise is made 11 ft. 4 in. and with diagonal as well as radial reinforcing rods, a 3-in. concrete thickness is adequate. By giving special attention to dome design larger tanks could be roofed in this way, however, the flat roof-slab construction lends itself readily to simple construction methods for the larger tanks and the supporting columns within the tank are not difficult to build nor do they constitute any serious disadvantage in operation.

The design of these tanks and the improved construction methods have been developed under the general supervision of J. S. Longwell, chief engineer and general manager, East Bay Municipal Utility District.

London subway 50 yr. old

The world's first tube railway celebrates its Jubilee last December. It is fifty years since Edward the Seventh, then Prince of Wales, inaugurated this pioneer line running south from the City of London under the Thames. Its original three miles of track is now two hundred, constructed at an average cost of £1,000,000 a mile. To-day it is operated by London Transport, serving a population of almost 10,000,000 people. One line, twenty five miles long, is believed to be the longest tunnel in the world.

From Field and Office

Working problems and time-saving methods for engineers and contractors

Tunnel Drilling Guide

Construction of Parker Dam powerhouse by the Bureau of Reclamation included drilling four 27-ft. circular penstock tunnels each about 150 ft. long with most of the tunnel sections on a 23-percent grade. To paint guides for the tunnel drilling, the unique method illustrated was followed.

After removal of the muck from a round, a station wagon with the specially built platform shown was backed up to the heading. On this platform was placed a set of pipe A-frames connected by a 1-in. sliding pipe. A 3-ft.-dia. plywood protractor was mounted on the 1-in. pipe so that it could be slid along the pipe and locked in position by a hand screw. The 18-ft. point pole had at its outer end an ordinary paint gun with air supplied by means of a welding hose attached to the pole, and a string fastened to the gun's trigger allowed the operator to start or stop the spray at will. A metallic tape attached to the pole gave the distance from the center of the protractor to the gun.

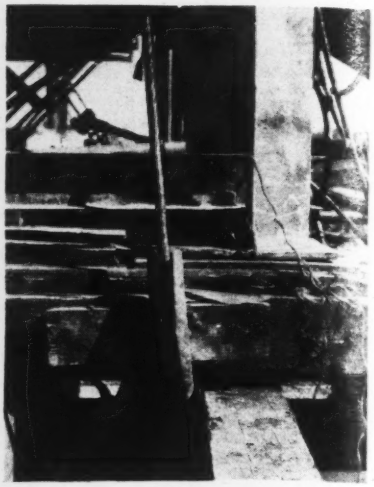
By moving the A-frame about or adjusting the legs, it was possible to quickly place the 1-in. pipe piercing the protractor on the tunnel's centerline. Sliding the protractor back and forth provided an azimuth in a plane normal to the center line as well as a guide for projecting the paint pole at right angles and for painting at constant radius from the center line. In sections where the heading was rough, the sliding protractor was especially advantageous.

In sections where the invert was too low to get the sliding pipe on the center line, polar coordinates were calculated.—TOM R. JOHNSON, *Associate Engineer, Bureau of Reclamation, Parker Dam, Calif.*

Drilling marker simplifies tunnel work for Parker Dam penstocks.

Cable Hitch Support

Wire lines used for dead-line hitches are apt to be permanently deformed if the hitch be made around an object less than the recommended diameter for sheaves adapted to the line being used. An eye for the hitch may be made by bending hot a length of 3-inch pipe, and then cutting away the outer portion of the curve to permit placing the wire in the shoe's curve. This method is much easier than trying to force the end of a stiff cable around the fairly sharp curve formed in a closed pipe. The pipe shoe may be slipped over or around whatever is being used as deadman for the hitch, the open end of the curve being ample to take in most types of supports used—M. T. PATE, *Houston, Tex.*



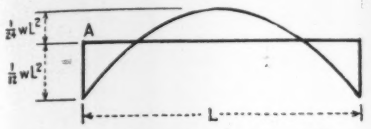
Reinforcing support prevents cable destruction and deformation.

Proportioning Connections For Fixed Beam Economy

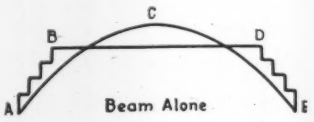
Since the maximum bending moment in a beam with fixed ends is generally less than that in a simply supported beam, economy may be secured in a steel structure by providing restraint for the ends of the beams. Greatest economy would of course result if the negative moments at the ends and the positive moment at the center were equal, but to effect this equality often requires flexibility in the end connections that is impracticable. Nevertheless, proper proportioning of the end connections does offer an opportunity for more economy than is generally achieved.

It is to be noted that the familiar diagram for a fixed end beam is not for the beam alone, but for the beam and connection. The connection carries the negative moment at the end and moment is transferred into the beam by means of the rivets in the flanges. The conventional moment diagram and the diagram for the beam alone are shown in the accompanying illustration. In the latter, point *B* represents the end of the connection. The maximum moment is at *B* or *C* and is much less than at *A* in the conventional diagram. It appears at once that there are interest-

ing possibilities in the moment diagram for the beam alone. A change in the length of the connection will move point B horizontally, while a change in its flexibility will move point B vertically. Since the maximum moment in the beam depends upon the location of point B, a very small value for maximum moment can be secured by careful design of the connection. As an example, if point B is made to fall at a distance of $\frac{1}{16}$ the length of the beam from the end, and the moment at B made



Beam and Connection

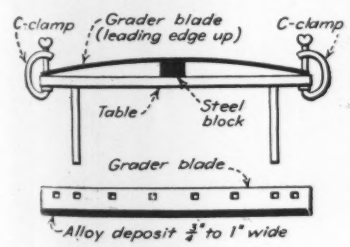


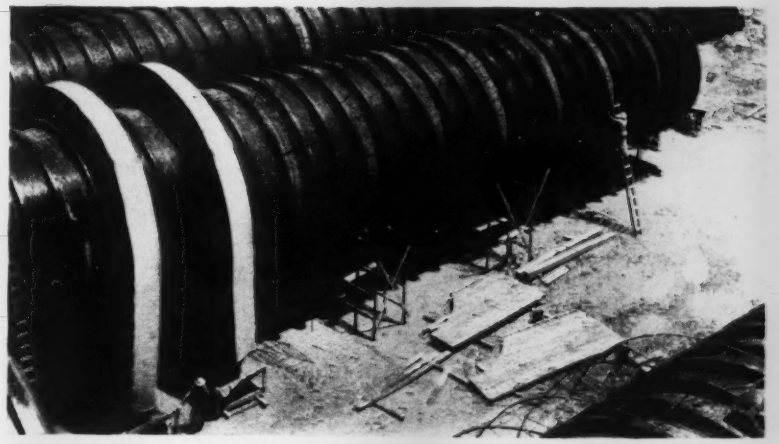
Beam Alone

equal to that at C, the maximum moment in the beam is $0.0478\ wL^2$ a substantial reduction from $1/8\ wL^2$. This result is based on the fact that the moment diagram for the beam without restraint is a parabola, and any condition of restraint can only affect the broken line ABDE.—CHARLES O. HARRIS, *Instructor in Mechanics, Armour Institute of Technology, Chicago.*

Hard-Facing Grader Blades

When hard-facing the cutting edge of a grader blade the possibility of warping is reduced by using a normalized or annealed 0.15 to 0.25 carbon steel blade and following the procedure indicated in the accompanying drawing. The blade is clamped to a bench, with the leading face upward, and over a steel block that gives it a crown. The height of the block should vary with the carbon content of the blade steel; for an 0.20 carbon steel a block $\frac{1}{2}$ in. thick is required for an





WELDED REINFORCING RODS *make scaffold for riveters and later serves as frame to hold canvas covers for recovery of material used in grit-blasting joints on the conduits for the outlet works at the Denison Dam.*

8-ft. blade. Thicker blocks are needed for steels having higher carbon contents. The hard alloy is applied to the blade edge for a width of $\frac{1}{4}$ to 1 in. and from $\frac{1}{16}$ to $\frac{3}{32}$ in. thick, using a blow pipe. When unclamped from the bench the blade straightens out so that it can be bolted to the mold board of the grader without warps. This information is from "Oxy-Acetylene Tips" published by the Linde Air Products Co., New York, N. Y.

F & O COMMENT

Plotting Points on Curves

Sir: In *Engineering News-Record*, Nov. 17, 1938, Thomas C. Coleman, Jr., gives a method of plotting points on curves. The following method which is probably simpler was evolved by the writer while engaged on the design of the Lake Ontario State Parkway for the Genesee State Park Commission. The method is particularly useful for plotting long curves on 50-scale maps. The procedure is as follows:

Tangent distance from P.C. *or* P.T.

$T1 = R \sin D$
$T2 = R \sin 2D$
$T3 = R \sin 4D$
$T4$ $R \sin 6D$ etc., where D = any number of degrees

Offset normal to tangents

$N1 = R$ versin D
$N2 = R$ versin $2D$
$N3 = R$ versin $4D$
$N4 = R$ versin $6D$, D as above

For example, assume a 3-deg. curve in which R = 1,910.08 ft.; then

Distance along tangent

$T1$ = 1910.08 × sin 4 deg. = 1,910.08 ft. × .06976 = 133.25 ft.
$T2$ = 1910.08 × sin 6 deg. = 1,910.08 ft. × .10453 = 199.96 ft.
$T3$ = 1910.08 × sin 8 deg. = 1,910.08 ft. × .13917 = 262.04 ft.
$T4$ = 1910.08 × sin 10 deg. = 1,910.08 ft. × .17365 = 331.68 ft.

Offset normal to tangent

$N1$ = 1910.08 × versin 4 deg. = 1910.08 ft. × .0024359 = 4.65 ft.
$N2$ = 1910.08 × versin 6 deg. = 1910.08 ft. × .0054781 = 10.46 ft.
$N3$ = 1910.08 × versin 8 deg. = 1910.08 ft. × .0097319 = 18.59 ft.
$N4$ = 1910.08 × versin 10 deg. = 1910.08 ft. × .0151923 = 29.02 ft.

By the above method any number of points can be plotted on the curve. Plotting a given number of points ahead from the P.C. and the same number back from the P.T., the offset points normal to the tangent locate the points on the curve. By varying the values of D the points may be plotted closely together or further apart as desired. There is nothing new in this as to geometry.—CHARLES S. SYNYARD, *Rochester, N. Y.*

Notes on Sewage Disposal

A monthly summary of current developments conducted by Willem Rudolfs

CONCRETE CORROSION—Tests and observations made during a number of years in the laboratory and field on the deterioration of concrete in contact with sewage has led Woolley (*Jour. Institution Civil Engineers*, England, Oct. 1940) to present a summary of important factors relating to such corrosion. Primary factors causing and abetting corrosion are considered to be: detention of sewage in pump wells; deposition of silt and sludge in sewers; seeding of fresh sewage with stale organic matter; agitation; condensation on submerged portions; summer conditions—including higher temperatures, lower flows and longer detention. The more important factors preventing and retarding corrosion are listed as: the use of pneumatic ejectors; total exclusion of air; complete access of air; increased ventilation; chlorination; use of special cement, paint or bituminous coatings.

SHORT-TIME DIGESTION—The general purpose of digestion is to produce an innocuous, rapidly drying sludge, greatly reduced in volume. The digestion time allowed at 80–85 deg. F. may vary from 25 to 40 days or even more. Results obtained over a period of a year at Manchester, England (*The Surveyor*, Dec. 27, 1940) with heated digestion tanks of about 110,000-cu.ft. capacity utilizing an average detention time of 12.1 days, are of interest. The main purpose of the plant is to deal with a maximum amount of surplus activated sludge, and by digestion and densification to reduce the volume to render it more amenable for disposal.

The proportion of primary tank sludge mixed with concentrated activated sludge has been kept as low as possible, consistent with the production of sufficient gas to maintain the required temperature in the tanks. During the summer time about 15 to 20 percent primary sludge is added, while in winter the quantity is increased to about 30 percent. Gas production is relatively low, amounting to 3.4 cu.ft. per pound of dry volatile matter added, but more than sufficient for the purpose since 1,885,000 cu.ft. of gas was burned as surplus. The pH values of the bottom sludge in the tanks varied from 7.2 to 7.6 with an average of 7.4, which is believed to have contributed largely to the complete absence of trouble due to foaming. The sludge added to the tanks had an average concentration of 3.15 percent solids which increased to 4.45 percent. Dry volatile matter was reduced 34 percent and the total volume of sludge 66 percent. The digested sludge is discharged at sea.

PRE-TREATMENT OF WASTES—Introduction of the "Trade Waste Act" in England has resulted in widespread discussion concerning the best methods and extent of pre-treatment. The same problems confront many local and state authorities in this country. Summarizing the experiences and discussions the nature of pretreatment can be divided into two groups (1) recovery of byproducts by the manufacturer; (2) pre-treatment to protect sewers and treatment plants by (a) regulation of flow, neutralization, temperature control, removal of coarse suspended solids, and (b) removal of a specific constituent to reduce the strength of waste.

Greatest difficulty is to decide how far pre-treatment is necessary. Small quantities of very toxic waste may not be detrimental when mixed with large quantities of sewage, while the same quantity of waste plays havoc with sewers or plant when only small quantities of sewage are available. Beside the technical difficulties the question of unequal treatment for different manufacturers, the possibility of antagonizing certain industries under such conditions, methods of dealing with wastes discharged in old connections and certain rights obtained by manufacturers when established, make the problem complicated and difficult.

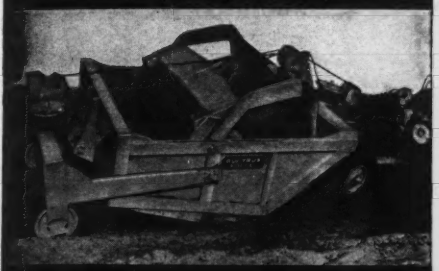
Hard and fast rules and regulations, considerations of safety and standards for sewage effluents, particularly in small towns where one or a few larger factories might alter completely the nature of the municipal sewage, have made the problem in England difficult. Hurley (*The Surveyor*, Nov. 1, 1940) enunciated these guiding principles: (1) as a rule wastes should be allowed in the sewers; (2) usually the degree of preliminary treatment demanded from the manufacturer should be the minimum necessary to afford reasonable protection to sewers and plant structures; (3) when the cost of pre-treatment is more economical than treatment at the plant, part or complete pre-treatment should be given at the factory; (4) every manufacturer discharging waste to the sewers should make payment for this facility with relation to volume and strength of the discharge; all old rights must be abolished; the problem cannot be equitably solved locally; (5) to remove disadvantages to inland towns, every manufacturer should contribute to a central fund an amount based upon the character and volume of wastes, so that payment would be on the same basis for any given type of industry. The local municipal authorities would be reimbursed in relation to the volume and degree of purification. If extra pre-treatment by the manufacturer is required he would also be reimbursed from the central fund.

PETROLEUM PRODUCTS furnish a source of energy for some species of bacteria and fungi. Total bacterial counts have been obtained with kerosene and mineral oil of more than 2 billion cells per cubic centimeter. The source of the organisms may be from water, intestinal tract of animals or disease processes of cow and horse. Bacterial decomposition of oils is comparatively slow.

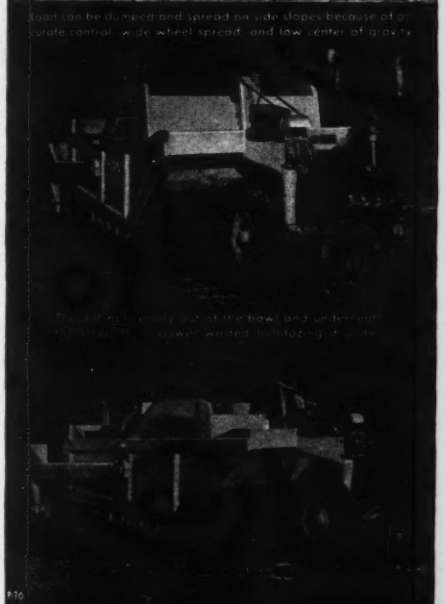
PHYSICAL METHODS of trade waste treatment, particularly ultra fine screening, are being used more and more, frequently in conjunction with chemical treatment. The American Stores Co. cannery plant at Hurlock, Md., packing asparagus, beans, tomatoes, beets, apple sauce, soups, uses a vibrating, fine-mesh screen of stainless steel cloth. The solids are used for fertilizer or hog feed.

Two BIG features

Side plates are extended well ahead of the cutting edge to eliminate side spilling of dirt while spreading or loading.



Load can be dumped and spread on side slopes because of accurate control, wide wheel spread, and low center of gravity.



Part of a load can be dumped, side plates and the rear

1 - Controlled Depth of Spread . . . . .

You get exactly the depth of spread you want with the new Bucyrus-Erie 4-Wheel Scrapers. As dirt is rolled out of the bowl it is leveled off by the cutting edge. Since this cutting edge is fixed in the frame and controlled by a separate line of cable, independently of the tilting of the bowl, it maintains the depth of spread you want at all times.

2 - Positive Rolling Ejection . . . . .

You also get positive rolling ejection, by means of the tilting bowl. Controlled by an independent line of cable, this bowl ejects the dirt in a rolling motion when tilted upward. It can be tilted to a nearly vertical position, and this sharp angle of tilt plus the shearing action along the side plates of the frame assures you clean dumping of any kind of material.

Of course the new Bucyrus-Erie scrapers have all the other features. There is a complete range of sizes matched to the new line of International TracTracTors. Find out all about them from your nearest International TracTracTor Dealer.

Bucyrus-Erie

SOUTH MILWAUKEE, WISCONSIN

CONTRACT UNIT PRICES

What contractors are bidding on different kinds of construction work

BRIDGE AND APPROACHES
MAINE

OWNER: Maine State Highway Commission, Augusta; L. D. Barrows, chief engineer.

PROJECT: Construction of Martin Point concrete, steel, double-leaf bascule span over Presumpscot River, between Falmouth and Portland, Me., on U.S. Route 1. Consists of 45 other spans in addition to bascule portion. Superstructure of steel and concrete. Treated timber pile bents. Total length of bridge 1,398 ft.; total project length 2,606 ft.; bascule section provides 75-ft. clear channel. Roadway 46-ft. wide; sidewalks 2½-ft. wide.

CONDITIONS: Contractor to furnish all materials and complete work by July 1, 1942. Rail, highway and water transportation facilities available. Minimum wages specified are: skilled labor, 80c. per hr.; semi-skilled, 50c.; and common, 40c.

BIDS: Six bids were received January 8, 1941, ranging from the contract low of $762,691 to $930,261.

LIST OF BIDDERS:

1. Wyman & Simpson, Inc., Augusta, Me. (contract) . . $762,691
2. Lane Constr. Corp., Meriden, Conn. 779,841
3. Ellis C. Snodgrass, Portland, Me. 842,101
4. Vulcan Constr. Co., Boston, Mass. 886,988
5. Carlo Bianchi & Co., Inc., Framingham, Mass. 913,502
6. W. H. Hinman, Inc., North Anson, Me. 930,261

Item	Quan.	Unit Prices (1)	(2)	(3)
1. Maintain traffic, 22-ft. rdwy	L.S.	$1,800.00	$9,500.00	$7,990.00
2. Cofferdams, 2 bascule piers	L.S.	35,000.00	50,000.00	63,114.00
3. Cofferdams, pier 1	L.S.	1,600.00	4,000.00	7,194.00
4. Struct. earth excav.; abut. and ret. walls	80 c. y.	1.00	2.00	2.00
5. Str. rock excav., abut. and ret. walls	10 c. y.	3.00	5.00	5.00
6. Str. earth excav., piers	5,100 c. y.	.55	1.25	4.00
7. Str. rock excav., piers	20 c. y.	20.00	15.00	20.00
8. Earth excav., rdway. and drainage	2,800 c. y.	.40	.90	1.00
9. Rock excavation, rdway. and drainage	1,000 c. y.	3.00	.90	5.00
10. Backfill	60 c. y.	1.00	1.50	1.00
11. Common borrow	2,500 c. y.	.46	.55	1.00
12. Gravel borrow	9,500 c. y.	.60	.90	1.15
13. French drains	50 c. y.	2.00	2.50	2.00
14. Riprap	3,650 c. y.	2.40	3.25	3.00
15. Loam	750 c. y.	1.20	1.25	1.30
16. Sodding	300 s. y.	.40	.60	1.00
17. Seeding, roadside mixture	40 M s. f.	5.00	5.00	6.00
18. Gravel base	3,300 c. y.	.60	1.10	1.15
19. Wire cable guard rail	1,600 l. f.	.40	.50	.35
20. Anchorages for above guard rail	7 ea.	10.00	10.00	9.00
21. Anchorages for guard rail at bridge	3 ea.	15.00	15.00	9.00
22. 12-in. corr. metal pipe	51 l. f.	1.30	1.50	2.00
23. 12-in. asph. coated corr. met pipe	154 l. f.	1.30	1.50	2.00
24. Drop inlets, regular	4 ea.	60.00	60.00	75.00
25. Drop inlet, special	1 ea.	130.00	35.00	25.00
26. Project markers	2 ea.	15.00	25.00	9.00
27. Right-of-way monuments	3 ea.	7.00	5.00	9.00
28. Conc.; abut., ret. walls, unwatered forms	155 c. y.	9.50	13.50	20.00
29. Conc.; piers, placed in unwatered forms	1,125 c. y.	8.00	15.50	12.00
30. Conc.; piers, placed under water	3,750 c. y.	9.00	6.50	6.00
31. Conc.; rdway., sidewalk slabs on steel bridge	1,896 c. y.	15.00	17.00	14.00
32. Reinforcing steel, delivered	683,000 lb.	.04	.03	.04
33. Reinforcing steel, placing	683,000 lb.	.006	.01	.01
34. Cement	15,400 bbl.	2.75	2.60	3.20
35. Cast iron drains	162 ea.	5.50	4.00	5.60
36. Piles, wooden, treated	65,840 l. f.	1.20	1.10	1.05
37. Piles, steel H-beams	24,900 l. f.	2.56	2.35	3.50
38. Struct. steel, riveted, del'd	834,000 lb.	.08	.08	.0772
39. Struct. steel, erection	834,000 lb.	.032	.025	.0291
40. Movable span machinery, del'd	190,000 lb.	.44	.38	.38
41. Erection, mov. span machinery	190,000 lb.	.023	.02	.02
42. Electrical equipment	L.S.	21,000.00	36,000.00	35,900.00
43. Hardware	20,000 lb.	.11	.10	.12
44. Gravel backfill	1,000 c. y.	3.10	2.25	2.00
45. Wrought iron for piers	182,800 lb.	.12	.12	.12
46. Waterproofing	30 s. y.	1.00	1.25	2.00
47. Remove existing structure	L.S.	1,000.00	2,500.00	6,000.00
48. Open grid floor	77,000 lb.	.11	.10	.0975
49. Counterweight concrete	88 c. y.	95.00	92.00	92.00
50. Sidewalk floor for basc. leaves	558 s. f.	2.30	2.25	2.75
51. Wrought iron rail	2,850 l. f.	8.60	7.00	6.65
52. Galv. wrought iron rail	293 l. f.	3.70	3.50	3.25
53. Traffic gates	4 ea.	460.00	400.00	400.00
54. Squared timber	73 Mbm.	128.00	125.00	140.00
55. Operator's house, complete	L.S.	1,800.00	3,000.00	2,102.20
56. Concrete roadway base slabs	1,090 c. y.	5.00	6.50	6.66
57. Bitum. conc. base course	200 ton	6.00	7.00	7.00
58. Tack coat	13,300 s. y.	.04	.03	.04
59. Bit. conc. surf. crse	14,500 s. y.	1.00	.70	.83
60. Door and windows, basc. piers	L.S.	300.00	275.00	383.00
61. Auxil. gasoline power	L.S.	1,270.00	1,100.00	1,100.00
62. Gray iron castings, bal. wts. and manhole frames	38,700 lb.	.063	.055	.06
63. Pile shoes	68 ea.	1.30	8.00	8.00
64. Pile points	1,146 ea.	1.30	.90	1.00
65. Str. steel, del'd., for appr. spans 1 to 8 and 10 to 46	1,290,000 lb.	.046	.045	.045
66. Erect. str. steel, spans 1 to 8 and 10 to 46	1,290,000 lb.	.011	.01	.0075
67. Underdrain	40 l. f.	2.00	1.50	.70
68. Conc. in basc. piers, pla. in unwat. forms; El.–20.5 and ctwght. pit	2,180 c. y.	8.00	8.00	8.00
69. Barrier gates	4 ea.	800.00	725.00	700.00

WATERWORKS IMPROVEMENT
ASHLAND, OHIO

OWNER: City of Ashland, Ohio; H. S. Lutz, mayor and director of public service. E. D. Barstow, Akron, engineer.

PROJECT: Construction of water softening plant, pumping facilities and water mains in Ashland, Ohio, an addition to existing plant and facilities. Involves installation of gas equipment, pumping equipment, flow control unit, valves, piping, etc. required for new building, and work on existing building and equipment to combine facilities.

CONDITIONS: Contractor to furnish all equipment and complete work in 200 calendar days. Rail and highway transportation facilities available. Average hourly labor rates are: skilled, $1.10 per hr.; semi-skilled, 80c.; and common, 60c.

BIDS: Seven bids were received November 29, 1940, ranging from the contract low of $51,490 to $68,533.

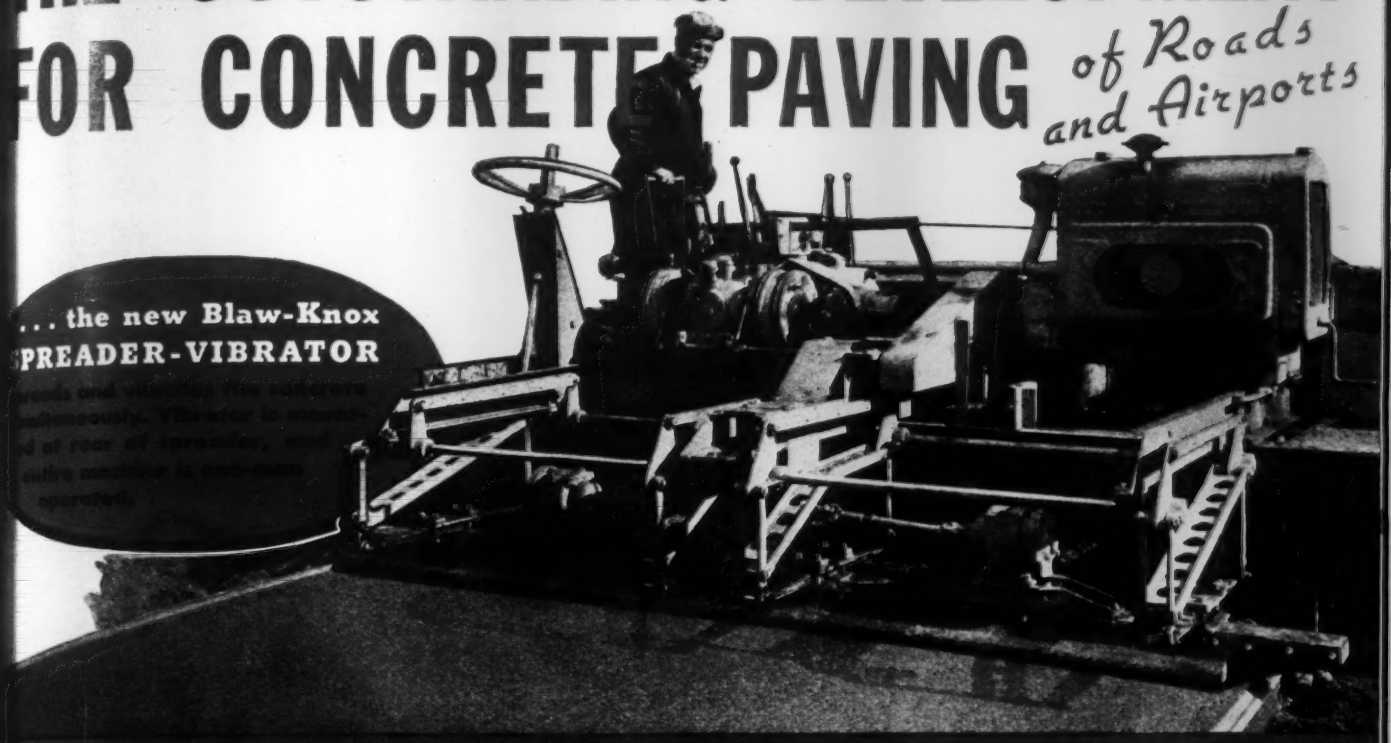
PARTIAL LIST OF BIDDERS:

1. Freeman and Jones, Inc., Cleveland, Ohio (contract) . . $51,490
2. Hanagan Bros., Urbana, Ohio 51,544
3. Steinle-Wolfe, Inc., Fremont, Ohio 53,699

Item	Quan.	Unit Prices (1)	(2)	(3)
1. Vitr. pipe sewer and drains	.....	$1,300.00	$2,108.00	$855.00
2. New building substructure	.....	7,200.00	6,184.00	6,290.00
3. New building superstructure	.....	2,800.00	3,536.00	3,194.00
4. Electrical work	.....	5,000.00	5,670.00	5,767.00
5. Heating and plumbing	.....	1,700.00	1,827.00	1,977.00
6. Carbon dioxide gas equipment	.....	2,300.00	2,563.00	2,169.00
7. New pumping equipment	.....	3,200.00	3,010.00	3,290.00
8. Move oil pumping equipment	.....	300.00	160.00	237.00
9. New rate of flow controller, etc.	.....	1,100.00	1,349.00	1,045.00
10. Inside valves and valve equip.	.....	500.00	502.00	535.00
11. Inside piping and appurtenances	.....	1,500.00	1,329.00	1,616.00
12. Work on existing building	.....	500.00	417.00	770.00
13. Work on existing controllers	.....	400.00	450.00	447.00
14. Work on exist. filter valves	.....	1,700.00	1,033.00	1,392.00
15. Work on exist. flow meter	.....	120.00	108.00	106.00
16. Work on existing main	.....	450.00	206.00	863.00
17. Underground c. i. and steel pipe	.....	3,800.00	3,190.00	4,311.00
18. Outside valves and appurtenances	.....	700.00	659.00	699.00
19. Valve vault, No. 2	.....	150.00	494.00	567.00
20a. Softening tank equip., type A	.....	11,500.00	10,946.00	11,686.00
20b. Softening tank equip., type B	.....	12,500.00	12,551.00	12,178.00
21. Chemical feed equipment	.....	1,100.00	1,330.00	1,290.00
22a. Extra concrete, class I	10 c. y.	20.00	22.00	27.00
22b. Extra concrete, class II	10 c. y.	15.00	20.00	26.00
22c. Extra concrete, class III	10 c. y.	10.00	16.00	25.00
23. Extra reinforcing steel	2,000 lb.	.05	.05	.045
24. Excavation, grading, etc.	.....	1,600.00	870.00	2,021.00
25. Gravel or slag refill	20 c. y.	2.50	4.00*	3.00
26. Work on exist. fillers	Force Acc.	500.00	500.00	500.00
27. Add'l. cleaning of manholes	3 ea.	100.00	201.00*	136.00
28. Emergency overflow	.....	220.00	267.00	287.00
29. Sludge lagoon and appurtenances	.....	2,400.00	1,873.00	2,850.00

* Calculated. Original bid on lump sum basis.

THE OUTSTANDING DEVELOPMENT FOR CONCRETE PAVING *of Roads and Airports*



BLAW-KNOX CONCRETE SPREADERS VIBRATORS ROAD FINISHERS

Introduction of the Blaw-Knox SPREADER-VIBRATOR marks a new trend in road paving methods—now, concrete of ½-inch and ¾-inch slump can be quickly and successfully compacted, surfaced and finished.

This new equipment, which provides an entirely new method for high speed, quality concrete paving construction, has aroused decided interest among highway engineers and contractors throughout the country. It's the most outstanding development in recent years for the paving industry.

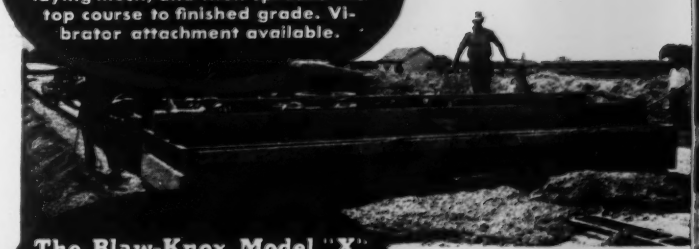
Mechanized Blaw-Knox equipment will build roads faster, cheaper and better. Get full information about Blaw-Knox Transverse Blade CONCRETE SPREADERS; SPREADER-VIBRATORS; Model "X" FINISHING MACHINES and VIBRATORY FINISHERS for wide or narrow roads.

●

BLAW-KNOX DIVISION of Blaw-Knox Company
FARMERS BANK BUILDING PITTSBURGH, PA.
NEW YORK CHICAGO PHILADELPHIA BIRMINGHAM
Representatives in Principal Cities

BLAW-KNOX CONSTRUCTION EQUIPMENT includes: Bins & Batchers • Road Forms • Street Forms • Concrete Spreaders • Tamping Rollers • Vibrators • Concrete Buckets • Clamshell Buckets • Truck Mixers • Striping Machines • Tractor Road Finishers • Road Forms

The Blaw-Knox Transverse Blade **CONCRETE SPREADER** automatically spreads the concrete dumped by the paver. It spreads for laying mesh, and then spreads the top course to finished grade. Vibrator attachment available.



The Blaw-Knox Model "X" **ROAD FINISHER** is designed for high speed production and quality finish. For both wide and narrow roads. Maximum 4-ft. width adjustability when specified. Vibratory attachment available.

MATERIALS AND LABOR PRICES

Market quotations on construction materials and wage rates reported monthly by ENR correspondents

LUMBER PRICES continue to be the most active in the construction material price list for March. Five cities report lower prices on 1- and 2-in. pine boards; one city is higher; and all others remain at February levels. Southern pine is $1.50 to $2.50 lower in Atlanta; $2.00 to $4.00 lower in Birmingham; off $2.00 in New York; and off $1.00 to $2.00 on 2-in. sizes in Chicago; but up 50c. to $1.00 in Detroit. Roofers North Carolina pine is off $2.00 on 1-in. sizes in Boston. Long leaf yellow pine is down $3.00 to $4.00 in Birmingham. Douglas fir is $2.00 lower in Chicago on most 2-in. sizes. Heavy timbers, however, do not follow the trend for smaller items, most of the price changes being upward.

Structural steel shapes are off 10c. per 100 lb. in Boston, but up 25c. and 30c., respectively, in San Francisco and Dallas. Expanded metal lath prices are mixed, higher in New York and St. Louis, but off in Boston and Cincinnati.

Portland cement is up 5c. per bbl. in Baltimore for the only change in the 20 cities reporting. Roofing materials are unchanged from February, but several manufacturers have published notice of price advances to be effective during March.

The Cincinnati bricklayers' rate which was $1.50 per hour, now ranges from $1.25 to $1.50; and plasterers' new rate ranges from $1.25 to $1.75, whereas the higher rate formerly prevailed. Common labor rate on building construction is now 67½c. to 87½c. in Cincinnati. The rate was 45c. to 87½c.

CEMENT, AGGREGATES, READY-MIXED CONCRETE—F.O.B. CITY

	PORTLAND CEMENT (Per bbl., carload lots, including 40c per bbl. for bags, cash discount not deducted)			SAND AND GRAVEL (Per ton, carload lots)			CRUSHED STONE (Per ton, carload lots)		CRUSHED SLAG (Per ton, carload lots, f.o.b. plant)		CONCRETE (Ready Mixed 1:2:4, 50 c.y. or more, delivered)
	Cloth Bags *a*	Paper Bags	Bulk	Gravel, 1½ in.	Gravel, ¾ in.	Sand	1½ in.	¾ in.	1½ in.	¾ in.	
Atlanta	$2.90	$2.66	$2.61	$1.75	$1.85	$1.95	$1.75	$1.85	$1.55	$1.65	$7.90
Baltimore	2.36	2.11	1.91	1.75‡	1.75‡	1.25‡	1.75‡	1.75‡	1.75‡	1.75‡	6.75
Birmingham	2.50	2.25	2.05	1.75	1.75	1.50	1.00	1.00	.90	1.15	5.80
Boston	2.75*b*	2.50*b*	2.30	1.20‡	1.20‡	.90‡	1.20‡	1.20‡	....	....	6.75*m*
Chicago	2.60	2.35	2.20	2.00‡*d*	2.00‡*d*	2.00‡*d*	2.00‡*d*	2.00‡*d*	1.25‡	1.25‡	....
Cincinnati	2.31	2.06	1.81	1.20	1.20	1.10	1.80	1.80	....	....	6.30
Cleveland	2.60	1.75	....	1.45*i*	1.45*i*	1.30*i*	1.70*i*	1.70*i*	1.70*i*	1.70*i*	6.55*i*
Dallas	2.47	2.22	2.02	1.75*p*	2.00*p*	1.25*p*	1.69*p*	1.84*p*	....	....	6.20
Denver	3.08	2.83	....	1.20	1.40	.90	....	....	....	....	7.25
Detroit	2.24	1.99	1.79	1.55	1.65	.95	1.55	1.65	1.30‡	1.40‡	6.50
Kansas City	2.56	2.31	2.16	1.85	2.30	1.00	1.91	1.91	....	....	7.25*n*
Los Angeles	2.25*b*	2.05*b*	....	1.00	1.00	1.00	1.00*f*	1.00*f*	....	....	5.85
Minneapolis	2.90	2.65	....	1.00*h*	1.00*h*	.25*h*	1.00*h*	1.00*h*	....	....	7.60*j*
Montreal	....	1.82*r*	1.67*r*	....	....	1.25‡	.85‡*c*	1.00‡*c*	....	....	6.45*l*
New Orleans	2.41	2.16	1.96	1.50	1.50	1.25	....	....	....	....	7.25
New York	2.55‡	2.30‡	....	1.25*de*	1.25*de*	.75*de*	1.65*de*	1.75*de*	....	....	6.50
Philadelphia	2.32	2.07	1.87	1.55‡	1.65‡	1.25‡	....	....	1.00	1.00	7.95*k*
Pittsburgh	2.44	2.19	1.99	1.40‡	1.40‡	1.70‡	2.25‡	2.25‡	1.50	1.50	7.50
St. Louis	2.55	2.30	....	1.50‡	1.50‡	1.25/1.50‡	1.00*h*	1.00*h*	.75*g*	.75*g*	7.50
San Francisco	2.16	1.96	1.66	1.36	1.36	1.36	1.46	1.46	....	....	6.95
Seattle	2.95*o*	2.70*o*	....	1.00*o*	1.00*o*	1.00*o*	2.00*o*	2.00*o*	....	....	7.25

‡ Delivered. *a* 10c. allowed for each returnable bag. *b* 10c. per bbl. off for cash. *c* Plus municipal tax. *d* Per cu. yd. *e* Barge lots alongside docks. *f* Crushed granite. *g* F.o.b. Granite City, Ill. *h* F.o.b. plant. *i* Within three miles of Public Square. *j* 5% discount for cash. *k* Discount 35c. 500 to 2000; 70c. 2000 to 5000; 95c. over 5000. *l* Up to 200 cu. yd. *m* 50c. off for cash. *n* 25 c.y. or more. *o* 2% off for cash. *p* 10c. per ton off, cash 15 days. *r* 10c. per bbl. off, cash 20 days.

CASH DISCOUNTS CEMENT to Contractors: truck delivery. 2% for cash on 10th of month; on carload deliveries, no trucking, same as dealer cash discount 10c. per bbl. for payment within 15 days of date of invoice. Subject discount 10c per bbl. 20 days in Montreal.

CURRENT MAXIMUM PRICES RECEIVED AT CEMENT MILL

Charge for bags not included. For cloth bags, add 40c. per bbl.; 10c. refund allowed for each returnable bag; for paper bags add 15c. per bbl., not refundable.

	Bagged	Bulk		Bagged	Bulk		Bagged	Bulk
Buffington, Ind.	$1.70	$1.65	Ironton, Ohio	$1.60	$1.55	Richard City, Tenn.	$1.80	$1 75
Dallas, Tex. (Inc. 5c. tax)	1.80	1.75	Limedale, Ind.	1.70	1.65	Steelton, Minn.	1.80	1.75
Hannibal, Mo.	1.70	1.65	Norfolk, Va.	1.65	1.60	Universal, Pa.	1.65	1.60
Hudson, N. Y.	1.75	1.70	Northampton, Pa.	1.55	1.50	Waco, Tex. (5c. tax in Tex.)	1.75	1.70
Independence, Kans.	1.70	1.65	North Birmingham, Ala.	1.70	1.65	Montreal	1.55	1.55

PAVING BRICK, BLOCKS, ASPHALT, ROAD OILS—F.O.B. CITY

	PAVING BRICKS AND BLOCKS			PAVING ASPHALT		ASPHALT BINDERS—FLUXES		CUTBACK ASPHALT		ROAD OILS	ASPHALT EMULSION (Quick-breaking)
	Granite per M. lots of 50,000 4x4x8 in.	Brick per M. 3¼x4x8¼ in. carload lots	Wood per sq. yd. 3½ in. 16-lb. treat	Per ton, less than 80 penetration		Per gal., 80-300 penetration		Per ton		Per gal.	Per gal.
				Tank car	Drums	Tank car	Drums	Tank car	Drums	Tank car	Tank car	Drums
Atlanta	$73.00	$39.35	$2.25	$17.30*e*	$23.83*e*	$0.0695*e*	$0.1018*e*	$0.0734*he*	$0.1257*h*	$0.0634*e*	$0.0734	$0.1257
Baltimore	160.00‡	45.00‡	2.75	14.00	20.00	.06	.09	.065*h*	.115*h*	.....	.....	.....
Birmingham	.....	27.50*j*‡	....	18.20	23.20	.0747	.1047	.0834*h*	.1484*h*	.....	.....	.....
Boston	85.00	47.50	3.00	14.00	20.00	.06	.09	.065*h*	.115*h*	.065	.09‡	.12‡
Chicago	135.00	45.00	1.85*d*	14.50	18.50	13.00*g*	17.00*g*	.06*h*	.10*h*	.04	.09	.125
Cincinnati	112.00	37.50	....	13.50	17.50	.07	.....	16.80	.....	.0525	.....	.075
Cleveland	110.00	37.50	2.25	15.00	20.00	.065	.075	.075*h*	.085*h*	.035	.08	.09
Dallas	.....	30.00	....	13.25	20.30	.06	.103	15.00	24.80	.055	.05/.07	.07/.09
Denver	.....	27.50	....	.....	.....	.....	.....	.....	.....	.....	.....	.....
Detroit	.....	35.00	....	13.15*f*	18.45*f*	.061	.0984	13.15	17.20	.0475	.061	.0923
Kansas City	.....	40.00	....	18.50	23.50	.077	.1075	.077*h*	.107*h*	.057	.11	.15
Los Angeles	.....	50.00	....	6.50*b*	15.00	6.50*bg*	15.00*bg*	5.50*b*	.....	.0192	.0525	.1125
Minneapolis	.....	.....	1.90*d*	19.20	24.20	.10	.12	19.30	28.30	.056	.....	.....
Montreal	75.00	.....	3.62	16.60	24.25	.116	.1483	.115*h*	.135*h*	.08	.135	.188
New Orleans	.....	.....	....	14.00*a*	19.00*a*	.055	.085	.06*h*	.09*h*	.....	.078	.....
New York	.....	.....	2.90	15.00	21.00	.06	.09	.065*h*	.115*h*	.....	.07	.11
Philadelphia	120.00	42.50	2.84	14.00	19.00	.055	.08	.06*h*	.09*h*	.05	.06	.09
Pittsburgh	135.00	38.75	....	.....	.....	.....	.....	.....	.....	.....	.....	.....
St. Louis	110.00/121.00	37.00	....	14.00	19.00	14.00*g*	19.00*g*	.06*h*	.095*h*	.04	.09	.125
San Francisco	.....	70.00	....	6.50	15.00	6.50*g*	15.00*g*	7.00	17.50	.025	.0525	.1125
Seattle	.....	60.00*c*	....	15.70	24.10	.....	24.10*g*	10.50/11.50	24.00/25.00	8.75*g*	.0575*i*	.1308*i*

‡ Delivered. *a* F.o.b. Baton Rouge. *b* Delivered to purchaser's warehouse. *c* 3¼x4x8¼ in. *d* 2½ in. 6 to 8-lb. treatment. *e* Local reduction due to 20% reduction intra-state class freight rates, only Georgia affected. *f* Mexican. *g* Per ton. *h* Per gallon. *i* F.o.b. Martines. *j* 3x3½x8½ in.

THIS NEW TRACING CLOTH *IS*

GHOST-PROOF!

Here is a new kind of tracing cloth that won't show perspiration stains or water marks—*that holds pencil smudges and erasure scars at a minimum.* Now you can have clean tracings, in pencil or ink, free from these untidy "ghosts" that reproduce on blueprints!

For PHOENIX is ghost-proofed by a remarkable new process that defies moisture, and gives you an unusually durable working surface. You can use harder pencils with this improved cloth and get sharper lines with less tendency to smudge. Even 6H pencil lines show clearly, and reproduce sharply! Erasing does not mar the drawing surface; erased areas take pencil smoothly—and ink without feathering. The new white color and increased transparency provide excellent drawing contrast and produce strong blueprints.

Try PHOENIX for yourself on your own drawing board. See your K&E dealer or write for a generous working sample and an illustrated brochure.

PHOENIX DEFIES MOISTURE GHOSTS . . .

Your hands are never dry. Perspiration stains ordinary tracing cloth, producing opaque spots, or "ghosts," that show on blueprints. Water splashes make even more disagreeable stains.

PHOENIX *Tracing Cloth withstands actual immersion in water for more than 10 minutes at a time without ill effects!* Perspiration will not stain it!

PHOENIX LESSENS SMUDGE GHOSTS . . .

The improved surface of PHOENIX Tracing Cloth permits you to use harder pencils (5H and 6H) and to get sharper lines with less tendency to smudge.

Result: cleaner tracings and blueprints!

PHOENIX REDUCES ERASURE GHOSTS . . .

Ordinary tracing cloths become scarred when erased. Erased spots produce ghosts on the blueprints.

PHOENIX has a durable drawing surface that reduces working scars to a minimum.

K&E

Phoenix

TRADE MARK

TRACING CLOTH

EST. 1867

KEUFFEL & ESSER CO.

NEW YORK • HOBOKEN, N. J.

CHICAGO • ST. LOUIS • SAN FRANCISCO • LOS ANGELES • DETROIT • MONTREAL

IRON AND STEEL PRODUCTS—BASE MILL PRICES

	STRUCT. SHAPES-PLATE	REINF. BARS ¼-in. billet *b*	RIVETS ½-in. structural	WIRE NAILS Base *d*	SHEET PILING Base	STEEL RAILS Per Gross Ton Standard	Light	Re-rolled	TRACK SUPPLIES Angle Bars	Std. Spikes *c*	Tie Plates *c*	Track Bolts
Birmingham	$2.10	$2.15	$3.40	$2.55	.....	$40.00	$40.00	$39.00	$2.70	$3.00	$2.15	$4.15
Chicago	2.10	2.15	3.40	2.55	$2.40	40.00	40.00	39.00	2.70	3.00	2.15	4.15
Pittsburgh	2.10	2.15	3.40	2.55	2.40	40.00	40.00	39.00	2.70	3.00	2.15	4.15
Buffalo	2.10	2.15	.....	.....	2.40	.....	.....	.....	.....	.....	2.15	.....
Cleveland	2.10	2.15	3.40	2.55	.....	.....	.....	.....	.....	.....	.....	.....
Youngstown	.....	2.15	.....	.....	.....	.....	.....	.....	.....	3.00	.....	.....
Detroit	.....	2.25‡	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
Gulf ports	2.45*a*	2.50*a*	.....	.....	2.85*a*	.....	.....	.....	.....	.....	.....	.....
Pacific ports	2.75*a*	2.55*a*	.....	.....	2.95*a*	.....	.....	.....	.....	.....	2.30	.....

‡ Delivered. *a* F.o.b. cars dock. *b* Rail steel same as billet prices. *c* Other basing points include Portsmouth, O., Weirton, W. Va., St. Louis, Kansas City, Minnequa, Colo., and Pacific coast ports, on tie plates, alone, Steelton, Pa., on spikes alone, Lebanon, Pa., Richmond, Va. *d* Add switching charge $18 per c.l.

IRON AND STEEL PRODUCTS—F.O.B. WAREHOUSE, PER 100 LB., BASE PRICE

	STRUCTURAL SHAPES Per 100 lb. base price	REINFORCING BARS* Per 100 lb., ¾ in., base price 15 tons or over *b* New billet	Rail steel	Add Switch	$/cwt. for Del.	EXPANDED METAL LATH—Per 100 sq.yd.. carload lots— Std. diamond mesh, 3.4 lb.	Std. ribbed 3.4 lb.	WELDED FABRIC REINFORCING—Per 100 s.f.. carload lots— 4x16 in., No. 5 & 10 wires	4x12 in., No. 8 & 12 wires	6x6 in. No. 6 & 6 wires Per sq. yd.	SHEET PILING Per 100 lb. base price
Atlanta	$2.34	$2.39	$2.30	.02½	.10	$21.00	$23.00	$1.82	$1.35	$0.1719	$3.04
Baltimore	3.70	2.90	2.75	.....	.10	24.00	26.00	1.70	1.27	.1611	2.90
Birmingham	2.10*a*	2.15	2.15	.02½	.10	20.50	22.50	1.80	1.34	.1701	.....
Boston	3.75	2.61	2.51	$20 C.L.	.10	15.50	17.50	1.76	1.31	.1656	2.77
Chicago	2.10*a*	2.15	2.05	.03	.05	20.00	22.00	1.67	1.25	.1575	2.40
Cincinnati	3.68	2.46*cd*	2.36*cd*	.....	.05	20.50	22.50	1.66	1.25	.1566	.....
Cleveland	3.58*g*	2.15	2.15	.02½	.10	19.50	21.50	1.66	1.25	.1566	.....
Dallas	4.90	3.80	3.80	.....	.....	22.50	24.50	1.93	1.42	.1827	.....
Denver	4.35	3.95	3.80	.....	.....	23.50	25.50	1.97	1.45	.1863	.....
Detroit	3.65‡	2.25	2.15	.....	.10	22.50	24.50	1.68	1.26	.1584	2.66
Kansas City	4.00	2.51*cd*	2.36*cd*	.0	.05	16.50	20.00	1.78	1.33	.1683	2.86
Los Angeles	3.65*f*	2.25	2.10	.02½	.05	23.00	27.50	1.93	1.42	.1827	3.65
Minneapolis	3.80‡	2.48*c*	2.48*c*	.0	.10	20.50/21.50	22.50/23.50	1.80	1.34	.1701	2.73*d*
Montreal	3.90	2.65	2.55	.....	.....	26.00*h*	33.00*h*	2.58	3.22	.264	2.58
New Orleans	4.40	2.52	2.52	.05	.10	21.50	23.50	1.82	1.35	.1719	2.875
New York	3.75	2.39	2.29	.....	.10	17.50	19.50	1.76	1.32	.1665	2.74
Philadelphia	3.55	2.66	2.56	.....	.....	19.50	21.50	1.71	1.28	.1611	2.72
Pittsburgh	2.10*a*	2.15	2.05	.02½	.10	21.50	24.50	1.59	1.20	.1503	2.40
St. Louis	3.69	2.39	2.39	.0	.05	24.50	28.50	1.69	1.27	.1602	2.70
San Francisco	3.75	2.50	2.35	.02½	.10	19.00	25.50	1.93	1.42	.1827	3.25
Seattle	2.75*e*	2.50*e*	.....	.....	.....	25.50	27.50	1.93	1.42	.1827	2.95*e*

‡ Delivered. *a* Mill prices. *b* 5–15 tons, add 15c. 1–5 tons, add 25c. Less than 1 ton, add 50c. *c* 20 tons or over Base. *d* Mill price plus freight. *e* F.o.b. dock. *f* Includes delivery in free delivery zone. *g* Less than 1 ton, add 30c.; 1 to 5 tons, add 10c. High scrap steel prices cut former 15c. differences between new billet and rail steel in many mills. *h* List price.

PLUMBING, HEATING, WATER, SEWER AND DRAIN PIPE

	C. I. PIPE Per net ton f.o.b. 6 in. to 24 in.*a*	VITRIFIED SEWER PIPE Per foot, std. 8 in., s.s.	delivered std. 12 in., s.s.	ASTM 24 in.,	C13–35 36 in.,	CLAY DRAIN TILE Per 1,000 ft., carload lots, f.o.b. 6 in.	8 in.	CONCRETE SEWER PIPE Per ft., delivered; ASTM 12 in.	C 14–35 24 in.	WROUGHT STEEL PIPE Full standard weight, *h* 1 to 3 in., Butt Weld Black %	Galv. %	3½ to 6in.; Lap Weld Black %	Galv. %
Atlanta	$49.40	$0.26*b*	$0.468*bc*	$1.8525	$4.68*c*	$85.00	$128.00	$0.45	$1.642	62.2	54.2	59.7	51.2
Baltimore	52.80	.40	.65	2.40	6.80	115.00	150.00	.60	1.90	65.5	57.5	63.00	54.5
Birmingham	45.00	.26	.468	1.7875	4.66	85.00	220.00	.40	1.60	62.2	54.2	59.7	51.2
Boston	55.00	.203	.4455	1.98	5.64	112.50‡	223.50‡	.45	1.69	64.1	56.1	61.6	53.1
Chicago	53.80	.28	.54	2.25	6.25	100.00‡	150.00‡	.38	1.48	67.0	59.0	63.5	55.0
Cincinnati	52.70	.196	.378	1.485	4.510	105.00	170.00	.60	1.70	65.9	57.9	63.4	54.9
Cleveland	54.24	.182	.351	1.395	4.305	69.00	108.00	.35	1.50	67.8	59.8	65.3	56.8
Dallas	58.20	.28	.504	2.016	4.70	80.00	100.00	.46	1.57	59.9	51.9	57.3	48.3
Denver	60.60	.24†	.52†	2.16†	.....	130.00	240.00	....	....	58.4	50.4	55.7	47.2
Detroit	54.24	.245	.4725	1.935	4.408	66.45	105.00	.33	1.50	66.3	58.3	63.8	55.3
Kansas City	55.12	.30	.54	2.16	5.00	120.00	240.00	.45	1.60	62.9	54.9	60.0	51.5
Los Angeles	55.00	.282†	.508†	2.034†	5.085†	115.50	192.50	1.00	2.00	59.6	51.6	57.0	48.6
Minneapolis	57.20	.24†	.432†	1.728†	4.60†	.....	.....	.70	1.80	64.2	56.2	60.7	52.2
Montreal	60.00	.55	.90	3.68	5.95*e*	135.00‡	260.00‡	.48	1.92	.....	.....	.....	....
New Orleans	51.38	.26	.48	1.69	.....	150.00	220.00	.37	2.10	62.4	54.4	58.8	50.3
New York	52.20	.32	.63	2.52	6.45	.....	.....	.50	1.20	64.9	56.8	62.4	53.9*i*
Philadelphia	50.90	.....	.....	.....	.....	.....	.....	.54	1.80	65.3	57.3	62.8	54.3
Pittsburgh	55.60	.245†	.4725†	1.845†	5.4325	97.80	154.20	.85*g*	2.00*g*	68.5*i*	60.5*i*	66.0*i*	57.5
St. Louis	52.48	.24	.432	1.728	3.96	85.00	170.00	.80*j*	1.60*k*	65.1	57.1	62.0	53.5
San Francisco	55.00	.3275*d*	.5895*d*	2.358	5.895	107.25	178.75	.54	2.50	57.6	49.2	54.8	46.0
Seattle	55.35	.35	.63	2.52	6.30	72.00	175.00	.35*f*	1.30*f*	.....	.....	.....	....

‡ Delivered. † F.o.b. *a* B. & S. class B and heavier, C/L lots, 200 tons and over Burlington, N.J. (base) $49.00. Gas pipe and class A, $3 per ton additional, 4 in., $3 per ton additional, 30 in. and larger usually $2 per ton less. *b* Double strength. *c* List to dealer. *d* List. *e* 30-inch. *f* Less 5% for cash. *g* Culvert pipe. *h* Discounts from standard list consumers carload prices, except Pittsburgh prices are f.o.b. mill. Base price $200 per net ton. List prices per ft.: ½ in., 8½c.; ¾ in., 11½c.; 1 in., 17c.; 2 in., 37c.; 2½ in., 58½c.; 3 in., 76½c.; 4 in., $1.09; 6 in., $1.92. *i* Applies also at Lorain, Ohio, mills. Chicago delivered base is 2½ points less on butt, 1½ on lap. Freight is figured from Pittsburgh, Lorain, O., Chicago Dist. Billing is from point producing lowest ptice at destination. WROUGHT IRON PIPE: Base price and list prices per ft. same as wrought steel pipe. Discount for Pittsburgh base: Butt-weld—1 in. and 1¼ in. black 34, galv. 19; 1½ in. black 38, galv. 21½; 2 in. black 37½, galv. 21. Lapweld—2½ in. to 3½ in. black 31½, galv. 17½ in.; 4½ in. to 8 in. black, 32½ galv. 20. *j* Reinforced; spec. C 76-37. *k* Reinforced; spec. C 75-37.

Let . . . "Hercules" (RED STRAND) Wire Rope Help You *Speed Up Production*

REG. U.S. PAT. OFF.

To maintain vital production schedules, it is highly important to use the kind of equipment that will give long and trouble-free service . . . *consistently.*

When it comes to wire rope, you can depend on "HERCULES" (Red-Strand). Proof of this is the fact that for over a period of 55 years it has been demonstrating its ability to meet the requirements of peacetime progress, as well as the demands of great emergencies. It has been able to do so because of the sound fundamental principles that have always governed its manufacture.

And this, too, is important. The same qualities and principles that make "HERCULES" (Red-Strand) Wire Rope so long lasting . . . so dependable, also make for greater economy.

In order to be suitable for all conditions, "HERCULES" (Red-Strand) Wire Rope is made in a wide range of both Round Strand and Flattened Strand constructions — all of which can be furnished either Standard or Preformed.

Leschen Rope is distributed from over 200 different points throughout the United States, which makes for quicker deliveries. Ask us for the name and location of our Distributor in your locality.

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WIRE ROPE MAKERS — ESTABLISHED 1857

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DENVER . . . 1554 Wazee Street

SAN FRANCISCO . . 520 Fourth Street
PORTLAND . . 914 N. W. 14th Avenue
SEATTLE . . 3410 First Avenue South

LUMBER, TIMBER, PLYWOOD—PER M FT., B.M., CARLOAD LOTS F.O.B.

	SHORT LEAF YELLOW PINE AND DOUGLAS FIR — All S. L. Y. P. is No. 2 common or better and for No. 1 N. C. Box. (Prices in Bold Face) — *All Fir planks No. 2 common, Fir timber is No. 1 common. Lengths up to 20 ft. (Prices in italics)*									LONG LEAF Y. P. Merchantable grade up to 20 ft.		PLYWOOD* Rail freight increment (See note for base price)	
	1x6 S4S	1x8 S4S	2x4 S4S	2x6 S4S	2x8 S4S	2x10 S4S	3x12 R/h	6x12 R/h	12x12 R/h	2x12 R/h	12x12 R/h	5/16"	3/8"
Atlanta	**$28.00**	**$28.00**	**$25.00**	**$26.50**	**$26.50**	**$29.50**	**$31.00**	.....	.....	$42.00	$55.00	$17.75	$21.60
Baltimore	**39.00**	**40.00**	**40.00**	**41.00**	**41.00**	**42.00**	**54.00**	.....	.....	78.00	80.00	17.35*g*	21.15*g*
	41.00	*41.00*	*48.00*	*48.00*	*48.00*	*48.00*	*54.00*	*$56.00*	*$56.00*	.....	.....	.....	.....
Birmingham	**29.00**	**29.00**	**27.00**	**27.00**	**27.00**	**30.00**	**48.00**	**48.00**	**52.00**	.....	.....	16.25	19.85
[1]	33.00	33.00	30.00	30.00	30.00	40.00	62.00	.....	.....	60.00	68.00	.....	.....
Boston	38.00[2]	38.00[2]	40.00[2]	45.00	60.00[2]	65.00[2]	80.00[2]	.....	.....	80.00	80.00	17.35*g*	21.15*g*
	38.00	*38.00*	*40.00*	*45.00*	*45.00*	*45.00*	*50.00*	*50.00*	*65.00*	.....	.....	.....	.....
Chicago	**42.00**	**42.00**	**38.00**	**38.00**	**38.00**	**43.00**	**52.00**	.....	.....	90.00	100.00	13.80	16.80
	40.00	*40.00*	*39.00*	*39.00*	*39.00*	*42.00*	*47.00*	*47.00*	*47.00*	.....	.....	.....	.....
Cincinnati	**37.50**	**39.00**	**38.00**	**38.00**	**38.00**	**41.00**	**50.00**	.....	.....	60.00	63.00	16.25	19.85
	.....	.....	.....	.....	.....	.....	*52.00*	*52.00*	*52.00*	.....	.....	.....	.....
Cleveland	**40.00**	**41.00**	**41.00**	**39.00**	**38.00**	**43.00**	**50.00**	50.00	50.00	83.00	83.00	16.70	20.40
Dallas *e*	**38.00**	**39.00**	**38.00**	**36.00**	**37.00**	**38.50**	**52.50**	**52.50**	**52.50**	**76.00**	**81.00**	**13.90**	**16.95**
Denver ‡	47.00[4]	46.00[4]	50.00[4]	50.00[4]	50.00[4]	52.00[4]	52.00[4]	64.00[4]	69.00[4]	.....	.....	9.90	12.05
Detroit	**42.50**	**42.50**	**41.00**	**42.50**	**42.50**	**44.50**	**51.00**	.....	.....	50.00	59.00	16.25	19.85
	.....	.....	.....	.....	.....	*49.00*	*52.00*	*59.00*	*59.00*	.....	.....	.....	.....
Kansas City	**41.50**	**41.50**	**38.00**	**38.00**	**38.00**	**41.50**	**47.00**	.....	.....	65.00	70.00	12.25	14.95
	42.50	*42.50*	*40.00*	*40.00*	*40.00*	*42.50*	*42.00*	*45.00*	*45.00*	.....	.....	.....	.....
Los Angeles *a*	*40.00*	*40.00*	*39.00*	*39.00*	*36.00*	*36.00*	*43.00*	*48.00*	*43.00*	.....	.....	9.35*gh*	11.35*gh*
Minneapolis ‡*b*	40.00[4]	40.00[4]	.....	.....	.....	.....	.....	.....	.....	.....	.....	12.05	14.70
‡	*42.00*	*42.00*	*42.00*	*40.00*	*40.00*	*42.00*	*46.00*	*53.50*	*53.50*	.....	.....	.....	.....
Montreal [7]	*36.00*	*38.00*	*36.00*	*38.00*	*40.00*	46.00	.....	.....	.....	155.00	165.00	.....	.....
	53.50	*53.50*	*54.25*	*54.25*	*59.25*	*59.25*	*65.00*	*65.00*	*65.00*	.....	.....	.....	.....
New Orleans	**24.00**	**25.00**	**25.00**	**24.00**	**24.00**	**25.00**	**46.00**	.....	.....	62.00	69.00	13.90	16.95
	29.00	*29.00*	*32.00*	*33.00*	*35.00*	*34.00*	*36.00*	.....	.....	.....	.....	.....	.....
New York ‡	**46.00**	**47.00**	**43.00**	**43.00**	**44.00**	**46.00**	**48.00**	.....	.....	85.00*f*	90.00*f*	17.35*g*	21.15*g*
‡	.....	.....	*52.00*	*52.00*	*52.00*	*52.00*	*54.00*	*54.00*	*54.00*	.....	.....	.....	.....
Philadelphia	**24.00**	**24.50**	**25.50**	**25.50**	**27.50**	**27.50**	**29.00**	.....	.....	50.00	57.50	17.35*g*	.1512*g*
	35.00	*36.50*	*39.00*	*38.25*	*29.00*	*39.00*	*44.00*	*32.00*	*32.00*	.....	.....	.....	.....
Pittsburgh	**50.80**	**53.20**	**62.75**	**55.00**	**60.85**	**62.50**	**76.25**	.....	.....	.....	.....	17.00	20.70
	69.20	*68.10*	*66.10*	*65.30*	*65.20*	*65.65*	*88.75*	*88.75*	*88.75*	.....	.....	.....	.....
St. Louis ‡*c*	**44.00**	**45.00**	**41.00**	**40.00**	**40.00**	**42.00**	**52.00**	*58.00*	*58.00*	75.00	74.00	13.90	16.95
San Francisco *d*	*28.00*	*27.00*	*28.00*	*29.50*	*29.00*	*29.50*	*29.50*	*28.00*	*28.00*	.....	.....	7.30*g*	8.90*g*
Seattle	*27.00*	*27.00*	*27.00*	*27.00*	*27.00*	*27.00*	*36.00*	*36.00*	*36.00*	.....	.....	0.00	0.00

Bold Face type, Southern Pine. *Italics, Douglas Fir.* [1] Long leaf. [2] Roofers' N. C. Pine. [3] Spruce. [4] Native. [5] Western Pine, No. 3 Common. [7] Spruce. [8] Norway Pine. ‡ Delivered. *a* Yard prices. *b* Contractors discount in Minneapolis and St. Paul discontinued May 21, 1938. *c* 5M ft. or less. *d* At ship's tackle. *e* 10% discount taken off. *f* Up to 18 ft.

NOTE: Special concrete form grade Fir Plywood, 4' by 8' panels, 5-ply, sanded 2 sides, water resistant glue, carload lots, delivered per 1,000 sq. ft. surface. Seattle base price on 5/16", $81.60; on 3/4", $89.90; price includes oiling and sealing charges. For other centers add rail freight increment from table or proper size. For resin dipped treatment, add $10.50 per M. *Many mills out of market due to rush of defense orders. *g* Lower rate by water shipment. *h* 50,000 lb. minimum

GLASS, EXPLOSIVES, CHEMICALS

	WINDOW GLASS — Discounts from jobbers list, Aug. 15, 1938 — Single or Double Thickness		EXPLOSIVES — Per lb. 40% Ammonia Gelatin in 50-lb. cases delivered in 200 lb. lots*
	A quality	B quality	
Atlanta	75%	75%	$0.15
Baltimore	82%	81%	.15
Birmingham	75%	75%	.105
Boston	78%	79%	.15
Chicago	79%	79%	.15
Cincinnati	77%	77%	.15
Cleveland	77%	77%	.19
Dallas	78%	78%	.1625
Denver	77%	77%	.155*e*
Detroit	79–10%	79–10%	.15
Kansas City	77–10%	77–10%	.155
Los Angeles	86%*d*	88%*d*	.1575
Minneapolis	76%	76%	.155
Montreal	40–10%	50–5%	.155†
New Orleans	70%	75%	.16
New York	81%	82%	.22*f*
Philadelphia	82–10%*d*	83–10%*d*	.15
Pittsburgh	79%	79%	.1225
St. Louis	77–10%	77–10%	.155
San Francisco	78%*d*	83%*d*	.155
Seattle	75%	76%	.1575†

Price of 60% Ammonia Gelatin is $0.015 per lb. higher than 40%

a Disc. from list Sept. 1939. *b* Also less 6% tax exemption. *d* Discount from jobbers' list Sept. 15, 1928.

* Urban prices influenced by service charges or local storage and delivery regulations, do not consistently reflect quantity prices in less congested areas. *e* F.o.b. Louviers, Colo. *f* In boroughs of Kings, Queens and Richmond, and in Manhattan south of Canal Street, add delivery charge of $6.00 per trip. † F.o.b.

40% Ammonia Gelatin price ranges in other than urban areas, per lb. (except Seismograph Grades)

	C/L 20,000 lb. net	Tons	200 lb. lots
E. of the Miss., except Fla	$0.105	$0.14	$0.16
W. of Miss. to Rocky Mtn. States, and Fla	.11 –.1175	.135 –.1475	.155 –.1675
Rocky Mtn. States	.105 –.1325	.125 –.1375‡	.135–.1525‡
Pacific N. W. States	.1075–.12	.1375–.15	.1575–.17
Pacific S. W. States	.1050–.1175	.135 –.1475	.155 –.1625

‡ F.o.b. Louviers, Colo., or Butte, Mont.

PILES, TIES—F.O.B.

PILES

Prices per linear foot, pine, with bark on, f.o.b. New York; delivered from barge 1½ to 2c. per ft. additional:

Dimensions	Points	Length	Short Leaf — Barge	Short Leaf — Rail
12 in. at butt	6 in.	30 to 50 ft.	$0.18	$0.195
12 in.— 2 ft. from butt	6 in.	50 to 59 ft.	.225	.24
12 in.— 2 ft. from butt	6 in.	60 to 69 ft.	.23	.255
14 in.— 2 ft. from butt	6 in.	50 to 69 ft.	.24	.265
14 in.— 2 ft. from butt	6 in.	70 to 79 ft.	.28	.275
14 in.— 2 ft. from butt	5 in.	80 to 85 ft.	.40	.415
14 in.— 2 ft. from butt	5 in.	85 to 89 ft.	.42	.482

RAILWAY TIES

Prices f.o.b., per tie for carload lots:		6"x8"x8' Untr.	6"x8"x8' Tr.	7"x9"x8'6" Untr.	7"x9"x8'6" Tr.
Boston	Oak	$1.20	....	$1.65	....
	Pine	1.50	$2.25*a*	2.25	$3.20*a*
New York	S. L. Sap Pine	1.20/1.30	1.95	1.60	2.45
	Mixed Oak	1.55	2.05	2.00	2.55
Birmingham	White Oak	.85	1.50*a*	1.35	1.70*a*
	Southern Pine	.55*b*	1.10*ab*	.70	1.30*a*
Chicago	Oak	....	1.65*a*	....	2.25*a*
	Southern Pine	....	1.53*a*	....	2.13*a*
Los Angeles	Douglas Fir	.64	1.38*a*	.90	1.80*a*
Philadelphia	Red Oak	1.10	....	....	2.40*a*
St. Louis	White Oak	1.20	....	1.60	....
	Red Oak	1.20	1.65*a*	1.60	2.20*a*
	Sap Pine or Cypress	1.10	....	1.50	....
San Francisco	Douglas Fir	1.00*bef*	....	....	2.45*caf*
Montreal	Birch or Maple	.65	1.05*a*	.75	1.35*a*

Tr.— Treated; Untr.— Untreated. *a* Creosoted. *b* 6" x 8" x 8'6". *c* Empty cell. *d* Zinc. *e* Green. *f* At ships tackle.

CHEMICALS

Water, sewage treatment, road work, f.o.b. carlots, New York

Bleaching powder, in drums, f.o.b. works, per 100-lb	$2.00–2.85
Chlorine cylinders, liquid, per lb. delivered	.0525
Calcium chloride, 77–80%, flaked, in 400-lb. drums or 100-lb. moisture proof bags, delivered, per ton	20.50–35.00
Silicate of soda, 52 deg., in drums, f.o.b. works, per 100 lb	1.40
Soda ash, 58%, in paper bags, per 100 lb. dense	1.10
Sulphate of aluminum, commercial, in 100-lb. bags, per ton	23.00
Sulphate of copper, in bbl., per 100-lb	4.75

STRUCTURAL CLAY BUILDING TILE, BRICK AND LIME—DELIVERED

	STRUCTURAL CLAY TILE — PARTITION — SCORED Per M. lots of 2,000 pieces or over, 3x12x12 in.	4x12x12 in.	8x12x12 in.	STRUCTURAL CLAY TILE — LOAD BEARING — SCORED Per M. lots of 2,000 pieces or over, 8x12x12 in.	10x12x12 in.	12x12x12 in.	BRICK Per M. in quantity. Common backing	Straight hard	LIME Per ton, Hydrated finishing	In paper, Common hydrated	Carload lot, Pulverized or lump
Atlanta	$90.00	$100.00	$185.00	$220.00	$230.00	$240.00	$12.50	$17.00	$26.50	$17.00	$17.00
Baltimore	75.00	80.00	150.00	180.00	225.00	280.00	14.00*k*	17.00*k*	18.50	15.00	2.50*m*
Birmingham	80.00	85.00	160.00	191.00	252.00	288.00	14.00	17.00	18.11	12.64	1.66*o*
Boston	98.70*a*	105.30*a*	206.85*a*	236.15	310.85*a*	346.10*a*	17.00*i*	20.00*i*	18.50	13.91	16.50*l*
Chicago	68.10	72.60	135.90	185.70	234.90	268.10	11.00	12.00	16.40	14.40	12.20
Cincinnati	61.40	65.30	121.90	141.50	187.30	215.20	16.00	16.00	15.90	13.50	.....
Cleveland	56.50	60.00	112.50	135.00	182.00	209.00	16.00	16.00	15.00*j*	12.00*j*	2.75*jm*
Dallas	50.00	52.00	100.00	135.00	200.00	250.00	11.50*i*	13.50*i*	19.00	15.00	.95*n*
Denver	84.80	94.40	152.70	178.20	212.00	227.90	13.50	17.00	30.00*j*	19.00*j*	30.00*j*
Detroit	69.00	73.60	138.00	176.00	232.00	265.00*e*	16.00	17.00	16.00	13.75	14.75
Kansas City	64.50	66.50	86.00*c*	125.50	159.75	186.25	14.50	24.00	19.10	12.80	10.70
Los Angeles	73.50†*r*	89.50†*r*	128.00*c*†*r*	210.00†*r*	368.00†*r*	460.00†*r*	13.50	15.00	16.50	.....	17.50
Minneapolis	71.45	76.20	143.95	171.45	173.80	242.10	13.10	15.10*g*	21.00	16.00	19.00
Montreal	72.20*a*	77.50*a*	142.50*a*	76.00*a*†	165.80*a*	248.70	17.50†	24.25†	23.50	17.00	12.00
New Orleans	72.00	72.00	135.00	162.00	210.00	260.00	14.00	.....	18.30	12.08	14.10
New York	87.00	93.00	128.00*c*	193.30*h*	238.80*h*	295.60*h*	13.00	.....	19.00	14.00	17.00
Philadelphia	80.50	85.90	161.10	193.30	238.80	295.60	16.00*e*	18.00*e*	15.70*q*	10.05*q*	12.05*q*
Pittsburgh	58.40	62.40	116.85	124.00	153.00	193.75	17.00	25.00	17.90	15.40	2.55*m*
St. Louis	62.00	88.00	120.00	180.00	234.00	280.00	16.00	17.00	22.00	16.00	2.30*lm*
San Francisco	84.00	94.50	225.00	.....	.....	.....	14.00	17.00	.55*o*	.40*o*	2.05*m*
Seattle	100.00	110.00	220.00	.....	.....	.....	18.00*f*	22.00*f*	20.00*s*	20.00*s*	20.00*s*

† F.o.b. *a* Smooth. *b* Carload lots delivered to job. *c* 6x12x12 in. *d* Not load bearing. *e* 48 lb. tile. *f* Less $1.00, ½ cash 15 days, balance 30 days. *g* Selected common. *h* F.o.b. Perth Amboy, N. J. *i* 50c. per M. off for cash. *j* LCL. *k* $1.00 discount if paid in 10 days. *l* Lump. *m* Per bbl., 180 lb *n* Per sack 100 lb. *o* Per bbl., 200 lb. *p* 280 lb. bag. *q* 25-ton cars. *r* 5% discount 10 days. *s* 2% off cash. *t* 5½ x 8 x 12.

PAINT, ROOFING—F.O.B. CARLOAD LOTS

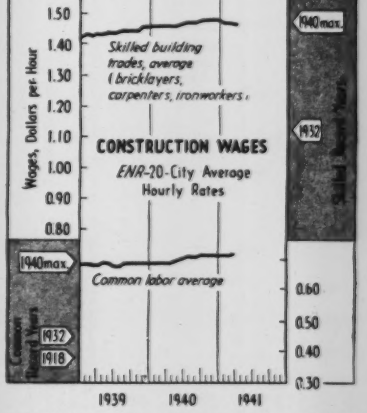
	RED LEAD Per 100 lb. in 600-lb. (Approx.) bbl. Dry *a*	WHITE LEAD Per 100 lb. in oil	READY-MIXED PAINT Per gal., drums Graphite *b*	Aluminum *c*	Ferric Oxide *d*	ROOFING SUPPLIES Carload lots f.o.b. factory Rolls, slate surfaced, 85-90 lb. per sq.	Asphalt felt, per 100 lb.	Tar felt, per 100 lb.	Asphalt coating per gal.	Tar pitch 350 lb. bbl. per ton
Atlanta	$9.25	$13.125	$1.30	$2.25	$1.20	$1.90	$1.61	$1.61	$0.27	$29.00
Baltimore	9.00	12.75	1.75	2.50	2.00	2.50	3.00	3.00	.60	29.00
Birmingham	9.25	13.125	2.35	2.75	1.85	1.98	1.57	1.57	.32	27.00
Boston	9.00	12.75	1.85‡	2.50‡	1.80‡	1.96‡	1.85*h*‡	1.85*h*‡	.50‡	20.00‡
Chicago	9.00	12.75	2.10	2.63	1.72	1.45*h*/*f*	1.50*h*/*f*	1.50*h*/*f*	.26/*f*	22.00/*f*
Cincinnati	9.00	12.75	1.65	2.85	1.55	2.40	2.39*h*	2.39*h*	.36	21.00
Cleveland	9.00	12.75	1.90	2.60	1.60	1.75	1.50*h*	1.50*h*	.35	21.00
Dallas	9.00	13.50	1.88	2.11	....	2.50*h*	2.60*h*	2.70*h*	.50	34.00
Denver	9.50	13.25	1.65	2.40	.17*l*	2.45	4.00	3.00	.35	28.00
Detroit	9.00	12.75	1.75	2.75	1.25	2.40	2.50	2.50	.35	26.00
Kansas City	9.00	12.875	2.15*g*	2.40*g*	1.67*g*	1.93	2.09	2.09	.25	25.00
Los Angeles	9.25	13.25	1.60	2.25	2.30	2.10	1.50	....	.45	.....
Minneapolis	9.00	12.875	....	....	....	1.75*i*	1.50*h*/*i*	1.50*h*/*i*	.29	21.00
Montreal	9.25	9.00	2.40	2.97	2.51	2.64	1.85	1.86	.63	1.85*k*
New York	9.00	12.75	2.02	2.55	1.64	1.75*	1.50*h**	1.50*h**	.23*	24.00*
Philadelphia	9.00	12.75	....	....	....	1.91*e*	2.05*he*	2.05*he*	.41*e*	22.00
Pittsburgh	9.00	12.75	1.05	1.95	.80	1.95	1.94*h*	1.94*h*	.34	26.00
St. Louis	9.00	12.75	1.90/2.00	2.60/2.80	1.60/2.00	1.75	1.50	1.50	.23	21.00
San Francisco	9.25	13.00	1.80	2.75	1.10/2.00	1.65	2.26	2.26	.28	17.00
Seattle	9.25	13.25	1.80	2.60	....	1.81	1.18*h*	1.75	.35	28.00/*j*

‡ Delivered. Note: Red lead in oil 50c. higher than white lead in oil *a* Red lead prices change frequently due to pig lead price changes. *b* U. S. War Dept. Spec. 3-49A. *c* ASTM Spec. D266-31. *d* 80% minimum ferric oxide. *e* Subject to 25% discount. *f* Distributors' price to contractors. *g* 5 gal. can. *h* Per roll, 65 lb. *i* Minneapolis and vicinity. *j* Asphalt pitch. *k* Per 100 lb. *l* Per lb. * Several manufacturers published notice of price advance to be effective during March.

SKILLED AND COMMON WAGE RATES—PER HOUR

	Bricklayers	Carpenters	Struct. Iron Workers	Hoisting Engineers	Plasterers	Common Labor Building	Common Labor Heavy Const.
Atlanta	$1.25	$1.00	$1.375	$1.00/1.25	$1.25	$0.40/.50	$0.40/.50
Baltimore	1.50	1.25	1.65	1.50	1.375	.625	.625
Birmingham	1.50	1.125	1.375	1.125	1.00	.40	.40
Boston	1.50	1.25	1.50	1.625	1.625	.60/.85	.60/.85
Chicago	1.70	1.625	1.70	1.70	1.70	1.025	1.025
Cincinnati	1.625	1.45	1.55	1.375/1.525	1.625	.75	.75
Cleveland	1.625	1.375	1.50	1.50/1.75	1.625	.90	.90
Dallas	1.50	1.00	1.25	1.00	1.50	.40/.50	.40/.50
Denver	1.65*b*	1.43*b*	1.43*b*	1.43*b*	1.50*b*	.718*b*	.718*b*
Detroit	1.50	1.25	1.625	1.50	1.55	.65/.75	.60/.775
Kansas City	1.625	1.375	1.50	1.50	1.575	.85	.50
Los Angeles	1.25	1.10	1.375	1.375	1.50	.625	.625
Minneapolis	1.375	1.25*d*	1.50	1.40	1.50*e*	.85	.85
New Orleans	1.50	1.25	1.375	1.375	1.40	.40	.50
New York	2.00*b*	1.85*b*	2.00	2.00*b*	2.00*c*	1.031	.875/.95
Philadelphia	1.65*b*	1.40	1.65	1.50	1.75	.70	.70
Pittsburgh	1.75	1.50	1.50	1.25	1.75	.80	.70
St. Louis	1.25/1.50	1.00/1.50	1.75	1.50/2.00	1.25/1.75	.675/.875	.875
San Francisco	1.75*c*	1.25	1.60	1.50	1.60*c*	.85/.875	.813
Seattle	1.60	1.25	1.50	1.50	1.60	.90	.90
Montreal	.80	.70	.75	.65	.80	.45	.40

b 7 hr. day. *c* 6 hr. day *d* 35 hr. wk. *e* 30 hr. wk.



ENR Skilled Average: (Bricklayers, Carpenters, Ironworkers) **$1.468**
ENR Common Average: $0.716

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PLYMOUTH
ROPE FOR INDUSTRY

MEN AND JOBS

T. C. MIDDLESWART, state district engineer of Nebraska, with headquarters in Bridgeport, has been transferred to Lincoln, in charge of district No. 1. R. A. HOUSE, who has been in charge of district No. 1, has been transferred to Bridgeport to succeed Mr. Middleswart.

MARSHALL V. JONES, construction engineer for the Nebraska Department of Roads, has been promoted to chief highway engineer. A. T. LOBDELL, former chief of roads and bridges, has been made administrative engineer. A. W. BOHNER, traffic engineer, has been promoted to construction engineer, and R. F. WELLER remains as assistant motor vehicle director. These appointments were recently announced by WARDNER G. SCOTT, state engineer at Lincoln.

DUDLEY R. BUCK, former chief PWA resident engineer for western New York state, at Rochester, has been transferred to Washington, D. C., to become examining engineer with the Bureau of the Budget. His duties will be inspection of defense projects in various sections of the country. Mr. Buck is a graduate of Ohio Northern University. JOHN H. O'SHEA, Buffalo engineer, will succeed Mr. Buck as WPA project supervisor in the western New York territory.

HARRY M. FREEBURN has resigned his position as district engineer for the Pennsylvania State Department of Health, to become chief engineer of the Philadelphia Suburban Water Co. He will be succeeded by CHARLES H. YOUNG of Meadville, Pa., as Philadelphia district engineer.

MAJ. WILLIAM G. SMITH, of Grove City, Pa., has accepted an appointment with the Government and has been assigned engineering duty at Borinquen Field, Porto Rico. He will have charge of construction of defense bases. Maj. Smith has been in charge of construction of institutional buildings for the past two years for the state of Pennsylvania.

H. E. BAILEY, of Perry, Okla., has been appointed city manager of Oklahoma City, succeeding W. A. QUINN who has resigned. Mr. Bailey is an engineering graduate of the University of Tennessee and has had 20 years experience as a practicing engineer. For the past two years he has been a member of the Oklahoma State Highway Commission.

R. W. HAZEKAMP, formerly engineer for the Chicago, Ill., park district, is now located in Washington, D. C., with the Civil Aeronautics Authority. He is associate airport engineer engaged on defense construction in cooperation with the Army and Navy.

The following have been appointed for an additional term as members of the Louisiana board of engineering examiners: DONALD DERICKSON, J. T. BULLEN, and B. H. GREHAN. Mr. Derickson is head of the Tulane University college of engineering.

HJALMAR WALBERG has been appointed Skagit County engineer at Mount Vernon, Wash. T. A. SWANSON, formerly engineer for the Union Oil Co., succeeds Mr. Walberg. W. C. WHITNALL, of Seattle, has been added to the Skagit county engineering staff as draftsman, inspector and transit man.

J. F. REYNOLDS and IVAN H. SMITH have formed the engineering and architectural firm of Reynolds & Smith Co., at Jacksonville, Fla. Mr. Reynolds is a civil engineering graduate of Iowa State College, Ames, Iowa, and Mr. Smith is a graduate of the University of Florida, School of Architecture.

LAWRENCE L. HALL, draftsman and project engineer in the Wyoming State Highway Department, at Cheyenne, resigned his position Feb. 15 to accept civil service employment as principal engineering draftsman with the U. S. Engineers, at Ketchikan, Alaska. Mr. Hall has been with the Wyoming Highway Department since 1929.

G. A. RIEDESEL has become acting assistant professor of civil engineering at the University of Idaho. Mr. Riedesel is graduate of Iowa State College. He is now on a year's leave of absence as resident engineer with the Idaho Bureau of Highways.

LOREN HEIPLE has joined the International Filter Co., Chicago, Ill., as service engineer. Mr. Heiple is a graduate civil engineer from Iowa State College, and holds a sanitary engineering degree from Harvard University.

M. J. HOFFMAN, state highway commissioner of Minnesota, has announced the appointment of E. V. H. BROWN as acting district engineer for the Rochester area. He succeeds T. C. COLWELL, engineer in the sixth district with headquarters at Rochester. Mr. Colwell is on active duty with the Army. EDWARD W. KUEGER, highway department engineer since 1922 and project engineer

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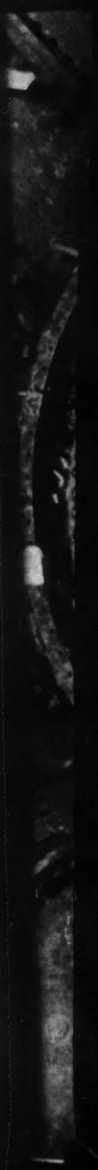
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